Novel Fatigue Analysis of Old Metallic Bridges through the Theory of Critical Distances (TCD)

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Abstract

Majority of the railway bridges built in the UK and around the world before the middle of 20th century are of riveted type made of wrought iron or old mild steel. Many of these bridges are approaching their useful design life. Despite their old age, many are still in operation. In-service fatigue cracking and failure in the riveted double-angle connections in such bridges, especially in the rivets and outstanding leg of the angles, had been reported in the past. These connections are one of the most fatigue-prone details found in riveted railway bridges.

Fatigue assessment of riveted details is generally based on a global life assessment, such as the S-N approach. The S-N approach involves the challenge of selecting an appropriate detail classification on an S-N basis as well as defining a nominal stress, free from stress raiser effects. By contrast, local stress methods, such as the Theory of Critical Distances (TCD), which directly considers the effect of stress raisers and loading conditions through finite element analysis, may provide a more favourable option for fatigue evaluation since it avoids the need for S-N classification and nominal stress calculation.

In this thesis, the fatigue life prediction capability of the TCD and the S-N methods when applied to riveted bridge details was investigated through finite element analysis of simple and complex riveted details focusing on stringer-to-floor-beam connections. The fatigue life predictions of both the TCD and the traditional S-N methods were compared with the experimental data. Thus, a database of static and fatigue tests available in the literature on structural wrought iron and mild steel riveted details and connections was created. The choice of the suitable modelling techniques for the finite element analysis of the riveted details was verified using a benchmark study on a riveted single lap joint. The critical length vs. number of cycles to failure relationship, \( L \) vs. \( N_f \), necessary for fatigue analysis based on the TCD was calibrated for structural wrought-iron material obtained from a real bridge. Fatigue assessment of the investigated wrought-iron material was performed based on both the TCD and the S-N methods to quantify the differences.

The results of this study showed that the TCD was successful in predicting fatigue life with the predictions falling within the constant amplitude scatter bands of the experimental data. In the single lap joint, the TCD method accurately estimated the average rivet clamping force values developed in the rivets of the specimens experimentally investigated. In the butt joints,
the results of the TCD method was found to conformed well with the experimental research in the literature by predicting higher rivet clamping forces in the specimens with longer grip length.

The novel formalisations of the TCD effectively predicted the fatigue life of full-scale riveted built-up girders with the results in the medium- and high-cycle always falling inside the CA scatter bands of the experimental data. The accuracy of the TCD method in estimating the rivet clamping forces present in the rivets of the investigated girders was found to be very high.

The TCD was highly accurate when used to estimate the fatigue life of different components of the stringer-to-floor-beam connection subjected to four-point bending with the predictions falling in the scatter of the available experimental data.

The hotspot locations identified by the TCD method were consistent with the results of the experimental study. By contrast, the predictions of the S-N method were very nonconservative in the case of the stringer-to-floor-beam connections (up to a factor of 2900).

In general, the S-N method predictions were found to be sensitive to the choice of the detail classification. In the case of the stringer-to-floor-beam connection, Modified Class B resulted in the most nonconservative results (up to a factor of 10) when compared with the results of the other S-N classifications. The findings of this thesis may provide the bridge owners and authorities with a safe and effective alternative method in determining the remaining fatigue life of such bridge details.
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Nomenclature

Roman lower-case letters

\(a\)  Crack length
\(da/dN\)  Crack growth rate
\(d\)  Rivet hole diameter
\(g\)  Gauge distance in double angle connection
\(m\)  Exponent for Paris law, slope of fatigue S-N curve
\(r\)  Distance measured from the point of maximum stress for any stress
\(t\)  Plate thickness, number of element used through the plate thickness

Roman upper-case letters

\(A, B\)  Constants of the \(L\) vs. \(N_f\) relationship
\(B\)  Specimen width
\(C'\)  Paris law material constants
\(E\)  Young’s modulus (Modulus of elasticity)
\(F\)  Geometry constant used in the equation for \(K\)
\(K\)  Stress intensity factor
\(K_2\)  Parameter defining the S-N relationship for two standard deviations below the main line
\(K_{ic}\)  Plain strain fracture toughness
\(K_t\)  Elastic stress concentration factor
\(L\)  Critical distance for high-cycle fatigue problems
\(L_M\)  Critical distance in the medium-cycle fatigue regime
\(L_S\)  Critical distance for static problems
\(N_f\)  Experimental number of cycles to failure
\(N_{fe}\)  Estimated number of cycles to failure
\(N_L\)  Number of cycles to failure delimiting the low-cycles fatigue regime
\(N_R\)  Design lifetime related to a constant stress range \((\Delta \sigma_R)\)
\(P\)  Applied point load, number of elements used around the rivet/rivet hole perimeter
\(P_{S}\)  Probability of survival
\(R\)  Stress ratio in cyclic loading
\(T_o\)  Scatter ratio of stress amplitude at 2 million cycles for 90% and 10% probabilities of survival
Nomenclature

Greek lower-case letters

\( \mu \) Friction coefficient
\( \sigma \) Stress, nominal stress applied to a notch or cracked specimen
\( \sigma_{\text{applied}} \) Applied remote axial stress
\( \sigma_{\text{nom},a} \) Amplitude of the nominal axial stress
\( \sigma_{\text{max}} \) Maximum nominal stress
\( \sigma_{\text{min}} \) Minimum nominal stress
\( \sigma_f \) Stress range in equations of design \( \sigma_f-N \) fatigue curves in BS5400-10
\( \sigma_{\text{ref}} \) Reference material strength
\( \sigma_S \) Inherent material strength
\( \sigma_y \) Yield strength
\( \sigma_{\text{UTS}} \) Tensile strength
\( \nu \) Poisson’s ratio

Greek upper-case letters

\( \Delta K \) Range of cyclic stress intensity
\( \Delta K_{\text{th}} \) Fatigue crack propagation threshold
\( \Delta K_c \) Stress intensity factor range for material toughness
\( \Delta \sigma \) Range of cyclic stress
\( \Delta \sigma_0 \) Fatigue strength of a plain specimen
\( \Delta \sigma_1 \) Maximum principal stress range
\( \Delta \sigma_{\text{ave}} \) Average stress range calculated according to the TCD
\( \Delta \sigma_c \) Reference value of the fatigue strength at \( N_c = 2 \times 10^6 \)
\( \Delta \sigma_D \) Fatigue limit for constant amplitude stress ranges at \( N_D = 5 \times 10^6 \)
\( \Delta \sigma_{\text{Exp}} \) Notched-specimen fatigue strength
\( \Delta \sigma_R \) Fatigue strength for constant amplitude nominal stress ranges

Symbols

\( \Phi \) Centre line
\( \Phi \) Rivet hole diameter

Abbreviations

2-D Two dimensional
3-D Three dimensional
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO</td>
<td>American association of state highway and transportation officials</td>
</tr>
<tr>
<td>AM</td>
<td>Area method</td>
</tr>
<tr>
<td>ASTM</td>
<td>American society for testing and materials</td>
</tr>
<tr>
<td>BS</td>
<td>British standard</td>
</tr>
<tr>
<td>C3D8</td>
<td>8-node three-dimensional linear brick element</td>
</tr>
<tr>
<td>C3D20</td>
<td>20-node three-dimensional quadratic brick element</td>
</tr>
<tr>
<td>C3D20R</td>
<td>20-node three-dimensional quadratic brick element with reduced integration</td>
</tr>
<tr>
<td>CA</td>
<td>Constant amplitude loading</td>
</tr>
<tr>
<td>CAFL</td>
<td>Constant amplitude fatigue limit</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon fibre reinforced plastic</td>
</tr>
<tr>
<td>COD</td>
<td>Crack opening displacement</td>
</tr>
<tr>
<td>DLJ</td>
<td>Double lap joint</td>
</tr>
<tr>
<td>FE</td>
<td>Finite element</td>
</tr>
<tr>
<td>FEA</td>
<td>Finite element analysis</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite element model</td>
</tr>
<tr>
<td>LEFM</td>
<td>Linear elastic fracture mechanics</td>
</tr>
<tr>
<td>LM</td>
<td>The line method</td>
</tr>
<tr>
<td>LP</td>
<td>Lower plate</td>
</tr>
<tr>
<td>MWCM</td>
<td>Modified wöhler curve method</td>
</tr>
<tr>
<td>PM</td>
<td>The point method</td>
</tr>
<tr>
<td>S8R</td>
<td>8-node doubly curved thick shell with reduced integration</td>
</tr>
<tr>
<td>SCF</td>
<td>Stress concentration factor</td>
</tr>
<tr>
<td>SLJ</td>
<td>Single lap joint</td>
</tr>
<tr>
<td>S-N</td>
<td>Stress-number of cycles</td>
</tr>
<tr>
<td>TCD</td>
<td>The Theory of Critical Distances</td>
</tr>
<tr>
<td>UP</td>
<td>Upper plate</td>
</tr>
<tr>
<td>VM</td>
<td>Volume method</td>
</tr>
<tr>
<td>UTS</td>
<td>Ultimate tensile strength</td>
</tr>
</tbody>
</table>
1 Introduction

1.1 Background

Wrought-iron and mild steel (i.e., steel produced during the late 19th and early 20th century containing less than 0.15% carbon content [1]) are the main materials used since the early 19th century in bridge construction in the Europe and America. Statistical data obtained through a survey regarding existing European railway bridges revealed that over 18,000 metallic bridges in the Europe are aged between 50 and 100 years old, and about 13,000 more metallic bridges are over 100 years old [2].

Damages due to the two world wars or excessive use of rail capacities because of an increase in freight volume demand have triggered several phases of repair or strengthening of these old bridges. However, very often, there is no evident sign of fatigue or deterioration. Cost of replacement as a result of reaching the design life for such a large number of old bridges, by far exceeds the available funds [2]. Therefore, a fatigue assessment method which can provide safe and accurate evaluation to prolong unnecessary repair or replacement of these old structures could be important for their owners from a safety and an economics point of view.

Further complications also exist when assessment of these old metallic structures is required since there is a large variation in their material characteristics due to the nonregulated manufacture of construction metal between 1860 and 1914. In case of steel bridge structures constructed during this period, fatigue is known to be the most important cause of in-service failure partly because it was not considered in the design process at the time [3]. Fatigue performance of different members and connections in riveted railway bridges is indeed the primary factor governing their life-span since these structures are subjected to repeated cyclic loading [3]. Fatigue damage is a complex progressive phenomenon affected by various factors such as, complex geometries, material defects or magnitude and type of loading. To avoid high costs of premature repair or replacement of these bridges or their fatigue susceptible members, accurate fatigue assessment methods are required. The nominal stress approach is a global fatigue design and assessment method adopted by bridge fatigue codes worldwide; this method has been shown to over- or underestimate fatigue life in case of connections with complex geometries, which can be found in older structures [4].
One of the main drawbacks of the nominal stress approach is the requirement for S-N classification of details; bridge details are classified according to a number of classification categories suggested by the bridge fatigue codes which cover typical details encountered in bridge structures. Most of these categories, however, cover modern bridge details such as welded and bolted details. Such S-N classification is challenging when it comes to old riveted bridge details since many of these details do not appear in the bridge codes’ databases. Even if they appear, one single classification is proposed to cover all riveted details found in such bridges.

Another drawback of the S-N method is the need to define a nominal stress to be used with the detail-specific design S-N curve for fatigue assessment. This nominal stress must be free from stress raiser effects and therefore creates additional challenges in case of complex details where the identification of nominal stresses is difficult.

The above reasons have raised the requirement for developing and utilising more capable approaches with better fatigue life prediction capabilities which explicitly consider the effects of stress concentrations and loading conditions, without the need for S-N detail classification and nominal stress calculation. In recent decades, developments in computer-aided numerical analysis has allowed the progress and application of refined local stress methods such as the Theory of Critical Distance (TCD) and the ‘structural hot-spot stress’ approach in fatigue assessment. The popularity of finite element analysis (FEA) is mainly because of its ability to produce a more detailed and accurate illustration of the stress state in complex details. The TCD method, which has been developed based on fracture mechanics, has been experimentally validated for a wide range of practical situations for various materials and notch shapes [5]. It should be added that using the TCD and hot-spot methods also has the advantage of avoiding the traditional FE problem of stress singularity or solution convergence. Stress singularity or convergence issues are situations where the solution does not improve when the mesh is progressively refined and the stress approaches infinity instead of reaching towards a finite value. These local analysis methods (TCD and hot-spot) use the stress value a certain distance away from the stress concentration point or potential stress singularity locations in the FE model and therefore can provide a reliable and accurate solution.

However, the TCD method has not yet been applied on a large scale for fatigue life assessment of fatigue-prone bridge details to quantify its accuracy against the conventional S-N approach.
There is, therefore, a lack of knowledge to ensure whether such methods could accurately perform fatigue life evaluation for simple and complex riveted bridge detail, as well as a lack of guidance for engineers about how to correctly carry out such fatigue assessments using FEA.

### 1.2 Research objectives

The main aim of this thesis is to verify the capabilities and applicability of the Theory of Critical Distances (TCD) method for estimating the fatigue life of complex riveted bridge details. A reliable fatigue life estimation is defined as one that falls inside the scatter of the experimental data or calculated fatigue strength with less than 20% deviation from the mean curve of the test results.

This research is one of the first to investigate the application of the TCD methods for fatigue assessment of simple and complex riveted connections. In an earlier study by Righiniotis et al. [4], the TCD methods were used to calculate fatigue damage to a stringer-to-floor-beam connection of a typical short span riveted railway bridge under the passage of BS5400 medium train traffic. The results were compared to that of the S-N method, and it was concluded that the latter can lead to overestimation of the fatigue life by a large factor. However, to the knowledge of the author, this is the first time that FE analysis of various types of riveted bridge connections has been carried out in such a scale to create a comparison between the fatigue life prediction capability of the commonly used S-N approach and the TCD method. Achieving such a target would fill the existing gap for a more reliable fatigue assessment method for old metallic riveted bridges.

To quantify the predictions made by the TCD, the traditional stress-based approach (also knowns as the S-N method), as a more commonly suggested fatigue design and assessment methodology by most codes of practice, has also been considered for fatigue life prediction. This has allowed for a better understanding of both these methodologies in terms of implementation and limitations.

The fatigue life of a large selection of details ranging from simple to complex has been evaluated to compare the equivalencies and differences of these two approaches. A database of common details encountered in riveted bridges, such as simple plates with hole(s), lap
joints, built-up girders and stringer-to-floor-beam connections, was collected from the technical literature. A finite element benchmark study has been performed on a single lap joint to increase the confidence in the modelling techniques used during the finite element analyses. Fatigue life predictions have been obtained using finite element models for all considered details through the TCD and S-N methods.

Within the context of this thesis, the primary objectives were therefore to:

- identify suitable FE element type(s) and optimal mesh densities capable of accurately capturing stress gradient in FE models which is required for fatigue life evaluation according to the TCD method.
- evaluate the accuracy level of the TCD as compared to the S-N method when applied to predict the fatigue life of simple details of riveted bridges.
- investigate the validity of the TCD method when used to estimate the fatigue life of riveted built-up girders obtained from old metallic bridges.
- determine the equivalencies between the fatigue life predictions based on the TCD and S-N methods for double angle connections (stringer-to-floor-beam connections).
- provide guidelines and framework for the implementation of the TCD method when used in conjunction with FE modelling for fatigue evaluation of fatigue-prone details of old metallic riveted bridges.

1.3 Scientific approach

The scientific approach considered in this research to achieve the main objectives is as shown in Figures 1-1 and Figure 1-2. A thorough literature survey was undertaken initially to study the TCD as a successful local fatigue analysis method and the S-N approach as a more commonly applied fatigue assessment methodology recommended in codes of practice. The available design S-N curves suggested in the British and European codes and standards for fatigue life evaluation of riveted bridges were the focus of interest. A database was created comprising of constant amplitude (CA) fatigue experimental tests on small- and full-scale details and components of riveted bridges collected from the literature.
Introduction

Figure 1-1 Scientific approach for fatigue life evaluation based on the TCD method

Figure 1-2 Scientific approach for fatigue life evaluation based on the S-N method
The TCD method suggests that an average stress range obtained near a stress raiser surface can be used in conjunction with plain material S-N curve (also obtained for the same stress ratio, $R = \sigma_{\text{min}}/\sigma_{\text{max}}$, as the one damaging the assessed detail) to estimate the fatigue life of the investigated detail. Different formalisations of the TCD can be adopted to determine this average stress; for instance, obtained at a distance away from the stress raiser surface (Point Method) or averaged over a certain length (Line Method), averaged over a critical area (Area Method) or a volume (Volume Method).

The TCD adopts a material-dependent characteristic length, $L$, in calculating the average stress range. The relationship between $L$ and the number of cycles to failure, $N_f$, for an old structural metal is estimated using the available relevant data found in the literature. This $L$ vs. $N_f$ relationship is required when carrying out fatigue evaluation according to the TCD method.

The subsequent step was to execute a mesh sensitivity analysis on a single lap joint to determine optimum mesh density and identify the effective FE element type(s) adequate for use in finite element analysis.

As can be seen from Figures 1-1 and Figure 1-2, the next step was to develop FE models of the simple and complex fatigue-critical details and components found in the literature review. Depending on the size of the FE model, different element type(s) and modelling techniques were adopted. For instance, in the case of large, simply supported girders, the shell-to-solid submodelling technique was incorporated for computational economy, while for the smaller models, only three-dimensional (3-D) brick elements were used to establish the FE models.

For fatigue assessment based on the TCD (see Figure 1-1), the relevant plain material S-N curve and the $L$ vs. $N_f$ relationship were employed to determine the fatigue life of the considered details by postprocessing the FE analysis results. Similarly, fatigue assessment per the S-N approach was also performed by using the design and assessment S-N curves suggested in various codes of practice for riveted structural details in conjunction with the nominal stress obtained from the results of the global FE analysis. In the final step of the scientific approach, the fatigue life estimated using the TCD and the S-N methods was evaluated and compared with the actual test data to quantify the differences and determine the degree of accuracy of both methodologies.
1.4 Outline of the thesis

This thesis consists of 10 chapters which are presented chronologically based on the adopted scientific approach. A brief description of each chapter content is as follows:

**Chapter 2 – Fatigue of steel structures**

This chapter includes a brief background on some of the commonly used global and local fatigue assessment methods.

**Chapter 3 – Fatigue assessment of old metallic bridges**

In this chapter, the S-N approach as suggested in the British standard, Railtrack codes of practice and Eurocode for fatigue assessment of riveted bridge details is presented. The Theory of critical distances as a capable local stress fatigue assessment methodology for fatigue life assessment is explained. Determination of the relationship between the critical length \( L \) and number of cycles to failure \( N_f \) for old structural metals used in fatigue life prediction according to the TCD in medium- and high-cycle fatigue regimes is also shown in this chapter.

**Chapter 4 – Fatigue tests of riveted bridge members and connections**

This chapter presents a short summary of the old and recent experimental fatigue tests obtained in the literature review on full-scale riveted bridge members and connections. The experimental tests on fatigue performance of stringer-to-floor-beam connections found in the technical literature are also presented in this chapter. The primary findings and main conclusions of these experiments are included in the description.

**Chapter 5 – A benchmark study on finite element modelling for fatigue assessment**

This chapter details the procedure and the results of a parametric study performed to identify the element type and optimal mesh density capable of accurately capturing the stress distribution at the vicinity of the rivet hole in a single lap joint subjected to eccentric longitudinal tensile uniaxial force.

**Chapter 6 – Fatigue life evaluation of simple details**

In this chapter, the TCD and S-N method are applied to predict fatigue life of simple details of riveted bridges. A total of eight cases are investigated, and the predictions of both methodologies are discussed and compared with the experimental data.
Chapter 7 – Fatigue life evaluation of riveted built-up girders
This chapter presents the estimates made when the TCD and S-N methods are used for fatigue assessment of riveted girders obtained from old metallic bridges. Fatigue life evaluation is performed on six wrought-iron or mild steel girders, and the results are compared to quantify the accuracy of both methods with regard to the test data.

Chapter 8 – Fatigue life evaluation of riveted stringer-to-floor-beam connections
This chapter gives the results of the fatigue tests on riveted stringer-to-floor-beam connections of the Vindelälven River Bridge obtained from the literature. It also investigates the applicability and validity of the TCD and S-N methods when applied to predict the fatigue life of such fatigue-prone connection subjected to four-point bending.

Chapter 9 – Conclusions and recommendations
In the final chapter of this thesis the primary findings of the study are summarised and main conclusions are drawn according to the evaluation of the obtained results. Some suggestions on the future work are also proposed.
2  Fatigue of steel structures

2.1  Introduction

Cast iron, wrought-iron and steel are the construction materials progressively used since the end of 18th century at specific periods in time. Structures such as buildings, bridges and industrial plants have been the main areas for usage of structural steel material. In [2], which is a survey geographically covering most of Europe and its major climates, it was concluded that the European metallic railway bridge stock is mainly constructed using mild steel and wrought-iron; about 70% are between 50 -100 years old with about 90% spanning below 10-40 meters in general. Result of another study on damaged steel structures covering the period between 1984 to 1995 revealed that bridges are one of the types of structures most susceptible to damage [3]. More than 62% of these damages occurred in less than 30 years after construction of the structure. As can be seen in Table 2-1 fatigue can be concluded to be the most frequently encountered cause of damage in steel bridge structures accounting for about 40% of the reported damage.

Table 2-1  Detail of the causes of damage and types of steel structures [6]

<table>
<thead>
<tr>
<th>Damage cause (Multiple denomination possible)</th>
<th>Totality</th>
<th>Buildings</th>
<th>Bridges</th>
<th>Conveyors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static strength</td>
<td>161</td>
<td>102</td>
<td>19</td>
<td>40</td>
</tr>
<tr>
<td>Stability (local or global)</td>
<td>87</td>
<td>62</td>
<td>11</td>
<td>14</td>
</tr>
<tr>
<td>Fatigue</td>
<td>92</td>
<td>8</td>
<td>49</td>
<td>35</td>
</tr>
<tr>
<td>Rigid body movement</td>
<td>44</td>
<td>25</td>
<td>2</td>
<td>17</td>
</tr>
<tr>
<td>Elastic deformation</td>
<td>15</td>
<td>14</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>Brittle fracture</td>
<td>15</td>
<td>9</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Environment</td>
<td>101</td>
<td>59</td>
<td>41</td>
<td>1</td>
</tr>
<tr>
<td>Thermal loads</td>
<td>23</td>
<td>23</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Others</td>
<td>5</td>
<td>2</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>Sum</td>
<td>543</td>
<td>304</td>
<td>128</td>
<td>111</td>
</tr>
</tbody>
</table>

In a similar study, which looked at the collapse of bridge structures made of all kinds of material in the past, fatigue cracks initiated due to secondary stresses or overloading were identified as the main cause of collapse of steel bridges [6]. 121 bridge collapse were reported during in service without any external influence. Poor workmanship and/or overestimation of the detail category were also among other contributing factors especially for welded structures.
Fatigue failure in railway bridges in terms of the modern codes of practice can be correlated to serviceability requirements. Design codes generally set permissible vertical deflection for railway bridges which could be related to safe working condition as well as passenger comfort. In EN 1990 [7] maximum permissible vertical deflections are given to limit the vertical acceleration during the travel inside the coach. The vertical deflection is a function of the span length \( (L) \), train speed \( (V) \), and number of spans and configuration of the bridge.

![Figure 2-1](image)

*Figure 2-1. Maximum permissible vertical deflection for railway bridges with 3 or more successive simply supported spans corresponding to a permissible vertical acceleration of \( b_v = 1 \text{ m/s}^2 \) in the coach for speed \( V \text{ [km/h]} \) [7]*

<table>
<thead>
<tr>
<th>No</th>
<th>Collapse</th>
<th>Number of collapses</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>With detailed info</td>
</tr>
<tr>
<td>1</td>
<td>During erection</td>
<td>93</td>
</tr>
<tr>
<td>2</td>
<td>In service without external influence</td>
<td>86</td>
</tr>
<tr>
<td>3</td>
<td>Due to ship impact</td>
<td>48</td>
</tr>
<tr>
<td>4</td>
<td>Due to influence of traffic under the bridge</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>Due to influence of traffic on the bridge</td>
<td>18</td>
</tr>
<tr>
<td>6</td>
<td>Due to high water level or ice</td>
<td>32</td>
</tr>
<tr>
<td>7</td>
<td>Caused by fire, explosion, etc.</td>
<td>15</td>
</tr>
<tr>
<td>8</td>
<td>Of the supporting framework</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>356</td>
</tr>
</tbody>
</table>

Table 2-2 Analysis of all known damages on bridges [6]
The vertical deflections should be determined with load model LM71 multiplied by the appropriate dynamic amplification factor (\(\varphi\)) excluding the adjustment factor ‘\(\alpha\)’ in accordance with EN1991-2 [8]. The deflection limits of the Figure 2-1 are for railway bridges with 3 or more successive simply supported spans and correspond to a vertical acceleration of 1 m/s\(^2\) for given travel speed. For different configuration of span a modification factor is suggested to alter the \(L/\delta\) values obtained from Figure 2-1. According to [9] the above method does not take into account track maintenance and requires dynamic analysis. In order that no dynamic analysis is required and to avoid the need for excessive track maintenance [9] recommends a simplified rule by using \(L/\delta = 800\) and \(L/\delta = (15V - 400)\) for \(V < 80\) and \(80 \leq V \leq 200\) km/h, respectively, calculated with LM71 and multiplied by \(\varphi\) with \(\alpha = 1\). Therefore, if the vertical deformation of the railway bridge due to fatigue crack(s) developed in members exceed values determined as above it may be considered as fatigue failure of the railway bridge.

Cost of replacement as a result of reaching design life for a large number of these old bridges, by far, exceeds the available fund. Therefore, a method of assessment which can adequately assess safety and durability of existing bridges is in the interest of bridge owners and authorities.

Fatigue can be described as a mechanism which covers an initiation and a propagation phase of cracks through the structural part as result of varying loading conditions. It is a time dependant progress caused by the damage accumulated from fluctuating stresses. Fatigue performance of structural steel is greatly influenced by many factors such as type and intensity of loading, environment, temperature, geometrical complexities, type of structural detail and material defects and impurities. The presence of local stress concentration features, where stress flow is disturbed, is the dominant factor governing fatigue crack initiation. These geometrical features can impose difficulties in accurate estimation of the load effects on fatigue life of structural members. In the case of large riveted steel structures, applying solely a global life assessment method may not result in adequately capturing the load effects on fatigue performance of complex details or connections. Application of a refined local stress fatigue assessment method which explicitly takes into account the effect of stress raisers and loading conditions may offer estimation of these effects with higher accuracy [10].

Significant improvement in computing field along with great development in numerical analysis during the last few decades has made the simulation of complex systems possible.
Moving away from simplified analysis and empirical rules towards accurate computer simulations has brought qualitative changes to engineering design. Employing finite element analysis (FEA) in fatigue design and assessment of bridges with complex geometries and loading has resulted in a more precise estimation of stresses in components compared to laborious task without using FEA. Accuracy of the FE model depends on the input knowledge such as applied loading, restrains as well as modelling skills to avoid excessive simplification which can result in undesired over- or underestimation of stresses. FEA still has some limitations with regard to size and complexity of modelled components, mainly when simulating non-linear behaviour and stress concentrations. However, magnitude of stress at which fatigue occurs in riveted bridges is normally well below the yield strength of the structural steel material. It is also known as high-cycle fatigue since it is related to fatigue lives above $10^5$ cycles and therefore fatigue assessment methods used generally only require linear-elastic stress analysis [7, 8].

### 2.2 Fatigue life assessment methods

Methods used in fatigue design and analysis of steel structures are generally based on: stresses, strains or stress intensity factors. There are a few commonly used assessment methods for fatigue life estimation of bridge structures which are briefly explained below.

#### 2.2.1 Stress and strain life methods

The stress-life (S-N) approach is an experimentally developed method which estimates the magnitude of peak stress (maximum tensile stress determined in the outermost edge of the cross-section using the elastic bending theory) in the fatigue-critical region and by employing S-N curve classifications attempts to estimate the number of cycles to failure. Studies done by August Wöhler, a German engineer, in the middle of the 19th century have originated this concept [12]. Results of tests on steel specimens subjected to different stress ranges ($\Delta \sigma = \sigma_{max} - \sigma_{min}$) under constant amplitude loading has generated these S-N curves which show that there is a linear relationship between the logarithms of the stress range and the number of cycles to failure, $N_f$, for any given detail. Data related to fatigue detail classes, that is, S-N curves, can be found in many design codes.
The strain-life (ε-N) method, also known as ‘local strain approach’, similarly uses the same principles, except that it takes advantage of the strains in the vicinity of stress concentration feature. This approach is considered in situations where fatigue loading will result in large-scale yielding in parts of the structure and localized plastic behaviour of high concentration sites. These two methods are practically identical. If the applied stresses are fairly low and in the region of elastic deformation, in practice, normally the S-N approach is adopted. Since both these methods are based on S-N curve data, they are not directly applicable if assessing complex details for which a fatigue design category is not provided [10]. Both these methods rely on an assumption that the material in the vicinity of the stress raiser feature behaves in a similar manner to the standard test specimens. For relatively large, blunt notches (where the notch root radius, ρ, is large enough to produce relatively small values of stress concentration factors, k), this is true while in case of sharp notches (smaller notch root radius where k tends towards infinity) or cracks (notch root radius of zero in magnitude) these methods can lead to major errors [5].

2.2.2 Linear elastic fracture mechanics (LEFM)

Fracture mechanics is another alternative approach for assessing stress concentration features taking into account crack growth rate in both short and long crack phases independently of S-N curves. Fracture mechanics show that the prediction of crack propagation is possible using linear-elastic analysis if well-defined conditions are in place. These methods are based on stress intensity factors, K, and hence, in order to calculate a stress intensity factor, an initial crack length has to be assumed. The stress intensity factor can be calculated using the following equation:

\[ K = F. \sigma \cdot \sqrt{\pi a} \]  
\text{Eq. 2-1}

where, F is the stress magnification factor (which is a function of loading conditions and crack geometry), \( \sigma \) is the remote applied stress and \( a \) is the initial crack length. Values of stress intensity factor, K, for different geometries can be found in the technical literature [13]. Some methods use the strain approach to estimate the number of cycles required to generate this initial crack while other methods assume this stage is negligible if the length of initial crack is
chosen to be small enough. When considering sharp cracks, these methods can provide appropriate estimates [5]. In fatigue problem cases, the range of stress intensity, $\Delta K$, is used to describe fatigue crack growth rate, as follows:

$$\Delta K = F \Delta \sigma \sqrt{\pi a} \quad \text{Eq. 2-2}$$

Fatigue crack growth rate ($da/dN$), which depends on the stress intensity range, $\Delta K$, is normally expressed in terms of crack growth amount per cycle. Figure 2-2 shows three phases that describe fatigue crack growth behaviour as a function of $\Delta K$ in log-log scale. It shows the existence of a material characteristic fatigue threshold value, $\Delta K_{th}$, below which crack propagation ceases or is extremely slow. Also, a critical upper limit exists, $\Delta K_C$, which is the fracture toughness of the material and for stress intensity ranges above that, a rapid acceleration in crack growth rate and fracture occurs.

![Figure 2-2](image)

*Figure 2-2 Typical fatigue crack growth rate curve*

However, in the region between the two limits, fatigue crack growth follows a linear stable rate which is governed according to Paris’ law [14]:

$$\frac{da}{dN} = C' (\Delta K)^m \quad \text{Eq. 2-3}$$

where $m$ and $C'$ are material constants derived experimentally for a given stress ratio, $R$. Increase of the stress ratio reduces the $\Delta K_{th}$ value and causes the crack growth rate curve
(shown in Figure 2-2) to shift left [11]. Nevertheless, these approaches are rarely used in bridge engineering practice because of the required extensive numerical modelling and the existing disagreement on the initial short-crack phase simulation procedure [5].

2.2.3 Structural hot-spot stress approach

The structural hot-spot stress approach is used for assessment of parent material adjacent to the weld toe while accounting for stress concentration effects of the overall geometry of a detail. The hot-spot method only assumes fatigue failure initiated from a weld toe and determines fatigue stress at critical points which are known as ‘hot spots’. The hot spot stress is derived from the FEA results using a linear two reference point system or quadratic surface stress extrapolation technique (as shown in Figure 2-3).

This leads to a mesh sensitivity disadvantage for this approach since FE model needs to be pre-processed with respect to post processing needs to allow creation of stress readout points at exact locations. Although hot-spot approach is a stress based method, it requires fewer number of S-N curves for evaluating number of cycles to failure, \( N_i \), which can be considered an advantage. This approach has been applied since 1960s for the fatigue design of pressure vessels and welded tubular connections. Improvement in computing power and finite element methods in recent decades has increased its application in fatigue life assessment of complex welded structures [10].
The Theory of Critical Distances (TCD)

The Theory of Critical Distance is not a single method but combination of a group of methods. When applied to fatigue problems, it assumes that fatigue damage can be estimated correctly if the elastic stress distribution ahead of the stress concentration feature damaging the fatigue zone is accurately known. If $\Delta\sigma_{\text{ave}}$ (average reference stress range used in the TCD for fatigue life estimation) which is dependent on the principal stress distribution ahead of the
Fatigue of steel structures

stress concentration apex equals to the plain material fatigue limit, the TCD assumes that the component is in its fatigue limit condition [15].

The TCD was first introduced in the 1930s in Germany by Neuber and in United States by Peterson in areas concerning fatigue failure perdition in notched metallic components. Later by the 1950s Neuber developed his ideas by invention of a theory so-called Line Method (LM) suitable for predicting high-cycle fatigue failure of notched components. Neuber believed that in situations of high stress gradient, application of classic theories which assume infinitesimal volumes for predicting elastic stresses were inaccurate. He suggested the use of a reference stress averaged over a critical distance from the apex of the stress concentration feature as a representative stress damaging the fatigue process zone.

A simplified version of this theory was later proposed by Peterson, that is now called the Point Method (PM), which suggests the use of a single value of stress as the $\Delta \sigma_{\text{ave}}$, at a given distance from the notch root. At that time, accurate estimation of elastic stresses near stress concentration features was not easily possible. This, along with the difficulty determining the critical distance value, led to empirical formalisation of the theories invented by Neuber and Peterson [4, 5].

Nowadays, advent of FE methods allows easier determination of the linear-elastic stress fields in complex 3-D geometries. This, in hand with taking full advantage of LEFM in calculating the critical distance value, $L$, has led to elimination of all the above challenges, allowing extensive application of the TCD as a capable engineering tool in fatigue life prediction analysis. When using the TCD, initially the material characteristic length, $L$, is to be calculated by linking the range of the threshold value of the stress intensity factor, $\Delta K_{\text{th}}$, and plain material fatigue limit, $\Delta \sigma_0$, both determined under the same stress ratio, ($R=\sigma_{\text{min}}/\sigma_{\text{max}}$ in stress cycle), with the one the assessed component is experiencing. $L$ in high-cycle regime is calculated as follow [5]:

$$L = \frac{1}{\pi} \left( \frac{\Delta K_{\text{th}}}{\Delta \sigma_0} \right)^2$$  \hspace{1cm} \text{Eq. 2-4}

The fact that the value of $L$ depends on two material properties, $\Delta K_{\text{th}}$ and $\Delta \sigma_0$, implies that $L$ can be assumed to be a material constant whose value is unique for different materials and stress ratios. It is important to keep in mind that for satisfactory predictions made using the TCD method, the value of $L$ has to be accurately evaluated using experimentally obtained
values of $\Delta K_{th}$ and $\Delta \sigma_o$ according to pertinent standards [15]. Once the value of $L$ is established, the subsequent step in the TCD method is the determination of the reference average stress range value, $\Delta \sigma_{ave}$, which is associated to $L$, using linear-elastic stress field in the vicinity of a stress concentration.

The PM is the simplest form of the TCD which assumes that a fatigue failure condition is reached if the range of maximum principal stress at a distance $L/2$ from the stress concentration root, along the focus path, is equal to the plain fatigue limit, as illustrated in Figure 2-4 (a). Focus path being defined as a line starting at the stress concentration tip and extending into the component in the likely direction of fatigue crack propagation [5, 12].

An alternative method to the PM is the modern formalisation of Neuber’s LM which uses the elastic stress field along the same focus path. The LM postulates that the assessed component is in its fatigue limit condition if the averaged maximum principal stress over a distance of $2L$ from the stress concentration apex equals to the plain fatigue limit, as shown in Figure 2-4 (b). The average stress range according to the LM criterion can be determined using Eq. 2-5 [4, 5]:

$$
\Delta \sigma_{ave} = \frac{1}{2L} \int_0^{2L} \Delta \sigma_1 (r) \, dr = \Delta \sigma_o
$$

\textit{Eq. 2-5}

where, ‘$\Delta \sigma_{ave}$’ is the average reference stress range, ‘$\Delta \sigma$’ is the range of maximum principal stress along the focus path, ‘$\Delta \sigma_o$’ is plain material fatigue limit and ‘$r$’ is distance along the focus pass. The reference average stress range damaging the fatigue process zone can be also calculated by averaging the maximum principal stresses which fall within an area (area method, AM) or a volume (volume method, VM) centred at the stress concentration tip. The area or volume shape used has an important effect on the results [5]. Bellett et al. [16] suggests that if a semicircular area or a hemispherical volume is used, the corresponding diameters of $1.32 \, L$ and $1.54 \, L$ should be used, respectively. It can be observed that in all forms of the TCD method, the same definition of $L$ can be used. In the case of complex 3-D geometries where determination of the direction of focus path is not as simple as 2-D problems and could result in conservative fatigue life predictions, VM may be a more reliable alternative to other TCD formalisations. Subsequent to determination of the average range of maximum principal stresses, in order to confirm whether the fatigue limit condition has been reached or not, this reference stress range is then checked against the constant amplitude fatigue limit (CAFL) of the plain material so that the effect of the stress ratio is...
taken into account. The plain material CAFL should correspond to the same stress ratio as the one damaging the assessed component [4].

\[
\sigma_{\text{nom},a} - \Delta \sigma_1 - \Delta \sigma_{\text{ave}} \\
\sigma_{\text{nom},a} - L/2 - r
\]

**Figure 2-4**  Linear-elastic stress distribution ahead of stress concentration tip along the focus path a) PM formalisation, b) LM formalisation [15]

A theory is of no value if it produces unreliable predictions and/or cannot be used in practical engineering situations. Validation of the accuracy of the above formalisations of the TCD has been broadly performed by various research studies through systematic comparison of analytical and experimental results confirming high-cycle fatigue strength predictions within a scatter band of about 20% compared to the experimental results [15]. These extensive checks were carried out considering standard notches such as blunt, sharp and short notches [17], circular holes [14, 15] and semicircular edge notches [19]. These investigations also included real engineering components such as car suspension arms [20], plates [13–15] and ship components [21]. Furthermore, it is interesting to observe that for notched specimens made of different materials like, for instance, titanium alloys [18, 19], cast iron [13, 15, 17], wrought-iron [4] and steel [14–16] the TCD was able to successfully predict the high-cycle fatigue strength even in the case of various values of stress ratios.

The TCD was also used in conjunction with the Modified Wöhler Curve Method (MWCM) to carry out fatigue life estimation of notched components subjected to constant and variable multiaxial fatigue loading [14, 20]. The MWCM takes into account the multiaxiality of the loading by considering the maximum shear stress amplitudes experienced on the critical plane where stage I (short crack growth propagation stage) of fatigue crack propagation takes
place. It assumes that fatigue damage also depends on the maximum normal stresses perpendicular to this critical plane. In order to include the effect of variable loading, the critical plane is defined as the direction experiencing the maximum variance of the resolved shear stress.

The TCD was confirmed to produce accurate results within the scatter bands of the experimental data used to calibrate the method itself for high- and medium-cycle fatigue regimes. In order to comply with the basis of the linear-elastic TCD, the length of the adopted critical distance, $L$, is taken as a material length parameter which increases as the number of cycle to failure, $N_f$, reduces in medium-cycle fatigue regime [24]. Finally it is worth mentioning that the TCD, when applied to estimate the high-cycle fatigue life of welded details, was capable of yielding very accurate prediction within 20% of the results obtained in experimental tests [21, 22].

Traditional stress methods, however, take into account the effect of stress concentration by using detail-specific S-N curves. This can become challenging when it comes to assessing complex geometries or details for which no specific S-N curve is available. The TCD on the other hand allows for stress concentration effects as well as secondary deformation-induced stresses by directly considering the linear-elastic stress field ahead of the stress raisers through FE analysis [4]. The use of only the plain material constant amplitude S-N curves in fatigue life assessment is another advantage of the TCD over the classical theories. It can be claimed that the TCD is a powerful engineering tool capable of accurate predictions as compared to traditional theories (Nominal stress approach and LEFM), and is applicable to a wide range of stress concentration features (blunt or sharp notches, cracks, welds, holes, etc.). Considering the limitations of LEFM and hot-spot method, along with its verified applicability to real engineering and structural components, the TCD has been investigated in this research study for fatigue life assessment of riveted railway bridges made of old steel material.

In the chapter that follows, the S-N approach as well as the TCD method as used for fatigue life assessment of riveted bridge details are presented. In case of the S-N method, the focus is on the method suggested in the British and European standards for fatigue life assessment of riveted bridge details. Additionally, different formalisations of the TCD method when adopted in conjunction with FE analysis for fatigue life assessment is also presented. The characteristic length, $L$, for old structural metal is determined using the data obtained from the technical literature.
3 Fatigue assessment of old metallic bridges

3.1 Introduction

One of the main objectives of this thesis is to assess the validity and applicability of the Theory of Critical Distance (TCD) in fatigue life estimation of old metallic riveted bridges. To assess the accuracy of the TCD and its applicability, the S-N approach is also utilised, as a more widely applied fatigue assessment method in steel structures, to create a comparative basis. Experimental fatigue data of some of the most commonly encountered simple and complex details and connections in riveted bridges have been collected from technical literature. These case studies are specifically selected where all the necessary information required to simulate the experimental analysis through linear-elastic finite element modelling are available to allow fatigue assessment based on the TCD methods. Comparison of the results estimated according to the nominal S-N and the TCD method with those of the experimental studies can quantify the reliability, effectiveness and efficiency of these fatigue assessment methods.

In this section, initially the S-N method as described in some of the commonly adopted codes of practice for fatigue assessment of riveted detail in steel structures, is explained. Subsequently, the TCD methods as applied to evaluate the fatigue life of riveted details constructed with old structural metal subjected to constant amplitude fatigue loading (CAFL), are described.

3.2 S-N approach (stress-life based method)

The magnitude of stress at which fatigue damage accumulates in old metallic riveted bridge structures is normally well below the yield strength of the structural steel material [12]. It is also known as medium- or high-cycle fatigue since it is related to fatigue lives above $10^5$ cycles. The S-N approach is the simplest and most commonly used method for high-cycle fatigue analysis and design of steel structures and forms the backbone of modern structural codes. The experimentally developed S-N approach is based on the nominal stress calculated using the linear theory of elasticity in the net section of the critical detail. In most design codes and guidelines, the S-N curves and detail classes associated with this approach, for a large range of structural details, are available.
‘Eurocode’ (BS EN 1993-1-9:2005) [27] does not specify any detail category for the assessment of riveted details. In [6], which is part of a background document to Eurocodes, Detail Category 71 of Eurocode is suggested to be used for the assessment of riveted structures, which is based on a 5% probability of failure for a 75% confidence level. This S-N curve has been determined by using the lower band of more than 125 full-scale fatigue test data for riveted details. For constant amplitude loading the fatigue life can be determined as follows [27]:

\[ N \sigma^m_R = K_2 \quad \text{with } m = 3 \text{ for } N \leq 5 \times 10^6 \quad \text{Eq. 3-1} \]

where \( K_2 \) is a constant defining the S-N relationship, for the mean line of the Detail Category 71 has a value of \( 1.88 \times 10^{12} \) and \( m \) is the inverse slope of the S-N curve. The reference value of the mean fatigue strength at \( 2 \times 10^6 \) cycles is 98 MPa. The constant amplitude fatigue limit below which an infinite number of cycles can be sustained, \( \Delta \sigma_0 \), is 72 MPa. This stress range is equal to the value of \( \sigma_R \) obtained using Eq. 3-1 for an endurance of \( N = 5 \times 10^6 \) cycles [27].

The British Standards for the fatigue assessment of bridges (BS5400-10) [28] suggests the use of the Class D design S-N curve for the purpose of fatigue assessment of riveted details and connections. The S-N curves in BS5400 are produced based on a 2.3% probability of failure and the number of cycle to failure for one given stress range (\( \sigma_r \)) can be determined using the following expression:

\[ N \sigma^m_r = K_2 \quad \text{with } m = 3 \text{ for } N \leq 10^7 \quad \text{Eq. 3-2} \]

where \( K_2 \) is a constant defining the S-N relationship, for the mean line of the detail Class D has a value of \( 3.99 \times 10^{12} \). The constant amplitude non-propagating stress range value, \( \sigma_0 \), for the Class D mean S-N curve is 74 MPa for \( N = 10^7 \) cycles, which implies constant amplitude stress ranges below this value are assumed to sustain an infinite number of cycles [28].

In [26, 27], it is suggested that the BS5400 fatigue curve B can also be used for fatigue assessment of riveted details. Since this class refers to the stress at the perimeter of a hole, the Class B S-N curve has to be divided by a stress concentration factor of 2.4, as suggested by BS5400, to allow it to be used in conjunction with nominal stresses. The new mean S-N
curve, termed as ‘modified Class B’, has a value of $K_2$ value of $7 \times 10^{13}$, a constant amplitude non-propagating stress range value, $\sigma_0$, of 51.5 MPa at $N = 10^7$ cycles and an inverse slope, $m$, of 4.

The UK Railway Assessment Code [31] provides recommendations for the parameters and methods to be used for the assessment of underbridges. It follows mostly the same principles as those used in BS5400-10. However, a few additional clauses and provisions for fatigue assessment of wrought-iron riveted structural details are included. An additional detail class is recommended for assessment of wrought-iron riveted details, termed ‘Class WI-rivet’. It follows the same expression shown in Eq. 3-2 to determine the number of cycles to failure with $m = 4$ for $N \leq 10^7$ and the $K_2$ value for the mean S-N curve is given as $9.73 \times 10^{13}$. For the Class WI-rivet mean S-N curve, the constant amplitude non-propagating stress range is 55.8 MPa at $N = 10^7$ cycles at location of rivets. Figure 3-1 shows the fatigue S-N curves for 50% probability of survival corresponding to fatigue design and/or assessment S-N curves suggested in [23, 25, 28] for riveted bridge details. As can be seen from Figure 3-1, the modified Class B and Class WI-rivet S-N curves are effectively identical and are then likely to create similar fatigue life estimates. The BS5400 Class D is the least conservative S-N relationship compared to the other curves presented in Figure 3-1 and therefore would be expected to give the least conservative fatigue life predictions.

In this study, the ‘mean’ S-N curves of Class D, modified Class B, Class WI-rivet and Detail Category 71 are used for the application of the S-N method for fatigue life estimation of the investigated case studies. The main reason for using the mean S-N curves rather than the design or assessment S-N curves suggested by the above codes of practice is to allow the TCD method to predict the mean value of fatigue strength as when the TCD method is used. Therefore, this would create a reasonable basis for assessing the accuracy of both S-N and the TCD methods in fatigue life evaluation of critical investigated details.
Figure 3-1 ‘Mean’ S-N curves (50% probability of survival) for fatigue assessment of old riveted bridge details subjected to constant amplitude loading as suggested by British and European modern fatigue design and assessment codes. The portion of the curves shown in dashed line represents the constant amplitude fatigue limit.

3.3 The Theory of Critical Distances (TCD)

As mentioned before, the S-N approach requires the selection of a detail category which best represents the investigated detail. In the case of old metallic riveted bridges this can become challenging since detail categories and classes provided by fatigue design and assessment codes normally cover modern details encountered in bridge structures, such as bolted and welded details. In some fatigue design codes, however, only a single detail classification is suggested to cover all types of riveted details that exist in bridge structures. Furthermore, defining a nominal stress value free of all the stress raiser effects to be used with this detail-specific S-N curve can be often challenging, especially for complex details under the influence of three-dimensional stress fields. On the other hand, the TCD approach, which is a local stress technique that explicitly takes into account the effects of stress raisers and complex loading conditions, may be a suitable alternative to overcome the above challenges. Moreover, the TCD method only requires the plain-material S-N curve and circumvents the challenging tasks.
Fatigue assessment of old metallic bridges

of detail classification and nominal stress calculation. This could prevent inaccurate fatigue life estimates because of the selection of a simplified detail class or nominal stress.

The TCD in the form of the Point Method (PM), Line Method (LM), Area Method (AM) and Volume Method (VM) is employed in fatigue life estimation in this thesis to allow comparison of these formalisations when applied in fatigue analysis of different riveted detail types. In the following sections, the TCD method for estimating the fatigue life of details subjected to constant amplitude loading (CA) is described in detail. It should be noted that the TCD method can also be modified to be applicable to fatigue life estimation of components subjected to variable amplitude (VA) or complex multiaxial fatigue loading.

3.3.1 Characteristic length, $L$ (critical distance) in constant amplitude loading (CA)

The TCD assumes that fatigue damage can be reliably estimated if the linear-elastic stress field ahead of the stress concentration root damaging the fatigue process zone is accurately known [15]. The fatigue process zone is that volume of material ahead of the stress raiser root within which the micro- or macroplastic fracture process takes place because of cyclic fatigue loading [32]. The TCD method enables estimating the fatigue life of a component using an average reference stress value, $\Delta \sigma_{\text{ave}}$, obtained a certain distance away from the apex of the stress raiser (PM), or averaged over a line (LM), or averaged over an area (AM) or within a volume (VM) [33]. The value of $\Delta \sigma_{\text{ave}}$ depends on a material characteristic length value, $L$, which, when addressing high-cycle fatigue problems, may be calculated as follows:

$$L = \frac{1}{\pi} \left( \frac{\Delta K_{\text{th}}}{\Delta \sigma_0} \right)^2$$

Eq. 3-3

where $\Delta K_{\text{th}}$ is the range of the threshold value of the stress intensity factor and $\Delta \sigma_0$ is the plain material fatigue limit both determined under the same stress ratio, $(R = \sigma_{\text{min}} / \sigma_{\text{max}} \text{ in stress cycle})$, as the one the assessed component is experiencing [34]. In order to utilise the TCD method in fatigue assessment of components containing notches and cracks in the medium-cycle fatigue region, the hypothesis is formed that, as the number of cycles to failure, $N_{\text{f}}$, decreases in the medium-cycle region, the material critical length value, $L_M$, increases [34]. In [33], the hypothesis is formed that $L_M$ vs. $N_{\text{f}}$ relationship in the medium-cycle fatigue regime
Fatigue assessment of old metallic bridges

is a simple power law, as shown in Eq. 3-4. This is because, in S-N curves, the number of cycles to failure and applied stress ranges are also linked through a similar power function:

\[ L_M(N_f) = A N_f^B \quad \text{Eq. 3-4} \]

where \( A \) and \( B \) (\( A > 0 \) and \( B < 0 \)) are assumed to be material parameters which have unique values for each specific material. It is recommended that these parameters are determined using a fully-reversed plain-material uniaxial fatigue curve as well as a fully-reversed uniaxial notched fatigue curve generated by testing specimens containing a known geometrical feature [30, 32]. In-field experience suggests considering stress raisers as sharp as possible for this purpose [36]. This is because sharp stress concentrations behave more in a cracklike manner and, hence, form a higher stress gradient, which makes it easier to determine the characteristic length value, \( L_M(N_f)/2 \), associated with the reference stress, \( \sigma_{ref} \) (see steps 5 and 6 of Figure 3-2).

As illustrated in the sketch in Figure 3-2, which shows the calibration process of the \( L_M \) vs. \( N_f \) relationship, it is straightforward to determine, for any number of cycle to failure, \( N_f \), the distance from the notch tip, \( L_M(N_f)/2 \), at which the magnitude of the corresponding linear-elastic maximum principal stress, \( \sigma_{ref} \), is equal to the amplitude of the stress which causes fatigue failure of the plain material at the same number of cycles \( N = N_f \) [34]. Repeating the procedure for several \( N_f \) values results in an accurate determination of the \( A \) and \( B \) constants and a reliable \( L \) vs. \( N_f \) relationship in the medium-cycle fatigue regimes. In Figure 3-2 the PM formalization of the TCD is demonstrated. Here, it is worth noting that similar calibration of \( L \) vs. \( N_f \) can also be performed using other formalisations of the TCD, such as the LM, AM or VM, with the only difference being the use of a more rigorous procedure when calculating the value of ‘\( \sigma_{ref} \). The linear-elastic stress field ahead of the stress raiser tip can be calculated using either FE analysis or analytical methods.
Fatigue assessment of old metallic bridges

3.3.2 Characteristic length for old structural metals in CA loading

A thorough literature survey revealed that very limited information is available on fatigue strength of plain and notched specimens from structural metals produced in the 19th century, that is, wrought-iron and early mild steel. There is no study available (to the knowledge of the author) which provides both plain and notched fatigue S-N curves for old structural metals as well as other necessary information required for fatigue assessment based on the TCD (such as, ΔKth) for the same stress ratio, R, as the one damaging the assessed specimen.

In [38], a series of direct stress fatigue tests were carried out on wrought-iron material taken from Brunel’s tubular bridge at Chepstow demolished in 1962 after 110 years of service life. Tension tests on the wrought-iron material from Chepstow Bridge generated an average ultimate tensile stress, σ_{UTS}, of 321.2 MPa, a yield stress, σ_y, of 222.4 MPa and a Young’s modulus of 188.4 GPa (average of 10 results). Fully-reversed fatigue tests were carried out under constant amplitude axial loading (CA) on the parent material and deliberately notched flat specimens (flat plates with one central circular hole). The original surface of the specimens was preserved to include the possible corrosion effects in the obtained results. A summary of the above test results is provided in Table 3-1.

Figure 3-2 Calibration of the L vs. N_f relationship in medium-cycle fatigue using plain and notched reference S-N curves and the linear-elastic stress field ahead of the stress raiser apex [37]
Table 3-1 Summary of the experimental test results generated for Chepstow Bridge material under static and fully-reversed CA loading

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>t (mm)</th>
<th>R</th>
<th>m</th>
<th>UTS (MPa)</th>
<th>T0</th>
<th>Δσ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>4</td>
<td>_</td>
<td>9.4</td>
<td>-1</td>
<td>11.3</td>
<td>296.5</td>
<td>1.28</td>
<td>230.8</td>
</tr>
<tr>
<td>Notched</td>
<td>6</td>
<td>14.2</td>
<td>14.2</td>
<td>-1</td>
<td>5.52</td>
<td>321.2</td>
<td>1.653</td>
<td>139.8</td>
</tr>
</tbody>
</table>

a Fatigue life at $2 \times 10^6$ cycles to failure related to net section area.
b Average plate thickness was used considering thickness of 10 different tested specimens.
c Averaged value of $\sigma_{UTS}$ for 10 tested specimens (upper limit = 353.7 MPa, lower limit = 278 MPa).

Figure 3-3 Results of CA fully-reversed axial fatigue test on Chepstow Bridge plain and notched wrought-iron specimens. Notched S-N curve refers to the gross cross-section area. (a) Detail of the notched specimens which gave a theoretical stress concentration factor, $K_t$, equal to 2.422 [38]

In [38], very few specimens were tested near the fatigue limit to establish an accurate constant amplitude fatigue limit. Thus, the above plain and notched fatigue curves for the Chepstow Bridge wrought-iron material are reproduced and presented in Figure 3-3 as single-slope S-N curves. These S-N curves are mathematically extended in positive and negative directions towards medium- and high-cycle fatigue regions ignoring that portion of the original curves that corresponds to the constant amplitude fatigue limit drawn in [38]. This was necessary to allow determination of the critical length, $L$, vs. the number of cycle to failure,
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$N_f$, relationship curve ($L-N_f$) required for fatigue assessment per the TCD methods. It is customarily acceptable in VA loading to extend the S-N curve below the fatigue limit either by the same or shallower inverse slope. However, since the exact position of the fatigue limit (knee point) is a tricky aspect, the S-N curve was extended at the same inverse slope in the high-cycle region assuming all stress ranges contribute to fatigue damage. It is shown later in this study that, this method of determining the $L-N_f$ relationship did not lead to inaccurate fatigue life estimations. However, it would have been more reassuring if adequate test data were available when estimating the $L-N_f$ relationship to reduce the requirements for such assumptions. The scatter bands shown by dash lines in Figure 3-3 refer to the 90% and 10% probabilities of survival, $P_S$, determined under the hypothesis that Log $N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) for a 95% confidence level. The scatter ratio of stress amplitude, $T_o$, for plain and notched specimens was found to be 1.28 and 1.653, respectively. $T_o$ is calculated by dividing the corresponding stress amplitude values at $2 \times 10^6$ cycles which intercept with the 10% and 90% scatter bands, respectively.

Figure 3-4 The $L$ vs. $N_f$ relationship for the Chepstow Bridge wrought-iron material determined according to the procedure described in Figure 3-2 using the PM formalisation of the TCD
The fatigue curves reported in Figure 3-3 were used to calibrate constants $A$ and $B$ of Eq. 3-4 according to the procedure described in Figure 3-2. The $L$ vs. $N_f$ relationship for the Chepstow Bridge wrought-iron material shown in Figure 3-4 was determined using the PM formalisation of the TCD. Abaqus/CAE v6.10 [39] was used to calculate the relevant linear-elastic stress fields in the notched plate. It should be noted that, to enable calibration of $L$ vs. $N_f$ in the medium-cycle region, the plain and notched fatigue curves were mathematically extended towards static failure condition using the negative inverse slope of the fatigue curves, $m$, given in Table 3-1.

The critical length value suitable for addressing static problems, $L_s$, shown in Figure 3-4 by a horizontal dashed line, was calculated using the following expression [36]:

$$L_s = \frac{1}{\pi} \left( \frac{K_{ic}}{\sigma_s} \right)^2$$  

*Eq. 3-5*

where $K_{ic}$ is the plain strain fracture toughness and $\sigma_s$ is the inherent material strength. For ductile materials, the value of $\sigma_s$ is generally larger than the ultimate tensile strength, $\sigma_{UTS}$, because of the plastic deformation that occurs before the total fracture of the specimen. However, for brittle materials, $\sigma_s$ approaches a value equal to $\sigma_{UTS}$ [33, 37].

The value of $K_{ic}$ for the wrought-iron material of the Chepstow Bridge is unknown. The extensive literature survey carried out by the author revealed that very little experimental data on plain strain fracture toughness of old structural metals is available. In the experimental tests performed by Brühwiler et al. [41], crack opening displacement tests (COD) were carried out on wrought-iron material of lattice girders obtained from a road bridge built in 1891 and replaced after 93 years of service life. COD tests were performed at intermediate loading rate (lower loading rates as compared to much higher rates used for standard Charpy tests, the so-called dynamic loading rate) to assess structural wrought-iron fracture toughness. For specimens tested at room temperature, a plain strain fracture toughness, $K_{ic}$, of 69 MPa$\sqrt{m}$ was obtained. The wrought-iron material tested in [41] has mechanical properties comparable to those of the Chepstow Bridge material, so it was deemed appropriate to assume similar values of $K_{ic}$ for the Chepstow Bridge wrought-iron.

Using Eq. 3-5 and assuming a $K_{ic}$ value of 69 MPa$\sqrt{m}$, the value of the static characteristic length, $L_s$, was calculated to be 14.7 mm. In this calculation, a $\sigma_{UTS}$ value of 321.2 MPa,
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provided in Table 3-1, was employed as $\sigma_S$. However, one can argue that since the Chepstow Bridge wrought-iron material demonstrated elongations between 8% and 39%, $\sigma_S$ could be higher than the average $\sigma_{UTS}$ used in the above calculation of $L_S$. Thus, assuming the upper limit value of $\sigma_{UTS}$ (353.7 MPa) for $\sigma_S$ from Table 3-1, gives an $L_S$ value of 12.1 mm. It can be observed in Figure 3-4 that, for number of cycles to failure $N_f < 20 \times 10^4$, the corresponding value of the critical length, $L_L$, becomes gradually greater than the $L_S$ values calculated for the static condition. This is taken as the low-cycle fatigue region for the Chepstow Bridge wrought-iron material.

The unique $L$ vs. $N_f$ relationship determined above (shown in Figure 3-4), especially for wrought-iron material, will be used throughout this thesis for fatigue life estimation of various simple and complex riveted details based on the TCD method. This will enable quantifying the accuracy of the TCD method as compared to the S-N approach which is a more commonly adopted fatigue assessment methodology in the British and European codes of practice (i.e., BS5400, Eurocodes 3 and Railtrack Codes of Practice).

The results of this study will also assess the use of a ‘general’ $L$ vs. $N_f$ relationship for fatigue analysis of old structural metals when the TCD method is employed. This is specifically of great importance since in many similar cases there is insufficient fatigue and static test data available to allow for determination of a material-specific $L$ vs. $N_f$ relationship. Moreover, in some cases, because of the old age of these structures (some being monuments and many being still in service), obtaining a sufficient number of samples to carry out static and fatigue tests is not always possible.

3.3.3 The TCD to predict fatigue life under VA fatigue loading

Considering the procedure explained earlier in Figure 3-2 for calculation of the critical distance, value of $L$ is expected to vary at different stress levels. In [36] it is proposed to use an equivalent critical distance, $L_{VA}$, in case of a component subjected to variable amplitude fatigue loading. A variable amplitude spectrum, as shown in Figure 3-5, can be presented as a series of stress amplitudes forming that spectrum. Imagine the notched specimen shown in Figure 3-2 subjected to the VA load history described in Figure 3-5 having $j$ different nominal stress levels with $n_i$ number of cycles for each stress amplitude. According to [36], each stress level of the VA spectrum can be treated as a CA sub-case. It is then possible, based on the procedure previously described for CA fatigue loading, to determine the number of cycles to
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Failure, \( N_{f,i} \) and the corresponding critical distance value, \( L_M(N_{f,i}) \) for each stress amplitude \( \sigma_{n,i} \) using either the PM, LM or AM. The \( L_{VA} \) can then be determined as shown in Figure 3-5. The \( L_{VA} \) is a weighted mean and its value would be dependent on the profile of the VA load spectrum and the amplitude of the nominal stresses involved.

\[
L_{VA} = \frac{\sum_{i=1}^{j} L_M(N_{f,i}) n_i}{\sum_{i=1}^{j} n_i N_{f,i}} = \frac{\sum_{i=1}^{j} L_M(N_{f,i}) D_i}{\sum_{i=1}^{j} D_i}
\]

**Figure 3-5 Proposed method for determination of the equivalent critical length, \( L_{VA} \), for variable amplitude fatigue loading [36]**

In the next stage, taking advantage of the calculated value of \( L_{VA} \), it is now possible to determine the magnitude of the reference stresses, \( \sigma_{ref} \), by applying either of the PM, LM or AM methods (see Figure 3-6) using the linear-elastic stress fields at the notch tip. Figure 3-6a shows the PM being employed in fatigue life estimation process, however, other formalisations of the TCD, that is, the LM, AM or VM may also be used to determine the \( \sigma_{ref} \) values. After obtaining the \( \sigma_{ref} \) values, the VA load spectrum can be recalculated in terms of the \( \sigma_{ref} \) amplitudes (Figure 3-6b) and for each resulting stress level it is possible to calculate the number of cycles to failure, \( N_{f,i} \), using the parent material S-N curve (Figure 3-6c). The total cumulative damage can also be directly calculated by summing the damage due to each stress level.

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3.3.4 The TCD to predict fatigue life under multiaxial fatigue loading

The Theory of Critical Distances (TCD) can also be adopted to determine fatigue lifetime in components subjected to the complex multiaxial cyclic loadings. In [34] the TCD method is used along with the Modified Wohler Curve Method (MWCM) to obtain fatigue lifetime of mechanical components subjected to multiaxial fatigue loading.

The fatigue strength at a stress concentration point depends on the complexity of the stress state at the vicinity of the hot-spot. The multiaxiality in fatigue problems can both arise from external complex loading condition or the internal geometrical features. The fatigue damage
caused in both scenarios could be similar as far as the stress field ahead of the notch apex is same. The MWCM is a critical plane approach which directly takes into account the degree of multiaxiality of the stress field since it is sensitive to both non-zero mean stresses and non-zero out-of-plane angles. The critical plane can be defined as the plane experiencing the maximum shear stress amplitude, \( \tau_a \). The MWCM assumes that the fatigue life is dependent on the maximum shear stress amplitude, \( \tau_a \), as well as the mean value, \( \sigma_{n,m} \), and the amplitude, \( \sigma_{n,a} \), of the normal stress relative to the critical plane. Based on the MWCM the fatigue behaviour of a material subjected to multiaxial loading regime can be described in terms of log-log S-N curves with the shear stress amplitude, \( \tau_a \), plotted on the y-axis against the number of cycles to failure, \( N_f \), on the x-axis. The degree of multiaxiality of fatigue loading for the S-N curves is measured using effective value of the critical plane stress ratio, \( \rho_{\text{eff}} \), and is defined as follows:

\[
\rho_{\text{eff}} = \frac{m \sigma_{n,m} + \sigma_{n,a}}{\tau_a}
\]

where \( m \) is a material constant representing sensitivity of the material to non-zero mean stresses normal to critical planes. The above expression shows capability of the ratio \( \rho_{\text{eff}} \) to take into account the effect of non-proportional loading and the mean stress in multiaxial loading. In Figure 3-7 the modified Wohler curves are represented in terms of both inverse slope, \( k_\tau (\rho_{\text{eff}}) \), and the reference shear stress amplitude, \( \tau_{A,\text{Ref}} (\rho_{\text{eff}}) \). The above figure shows that an increase in the ratio \( \rho_{\text{eff}} \) results in a reduction in the fatigue strength for a given shear stress amplitude, \( \tau_a \). Using a fully reversed plain material uniaxial and torsional fatigue S-N curve it is possible to determine the relationships between the \( k_\tau \) versus \( \rho_{\text{eff}} \) and \( \tau_{A,\text{Ref}} \) versus \( \rho_{\text{eff}} \). These relationships can be formalised as follows:

\[
k_\tau (\rho_{\text{eff}}) = a \rho_{\text{eff}} + b \quad \text{Eq. 3-7}
\]

\[
\tau_{A,\text{Ref}} (\rho_{\text{eff}}) = \alpha \rho_{\text{eff}} + \beta \quad \text{Eq. 3-8}
\]

where \( a, b, \alpha \) and \( \beta \) are material constants obtained from the experimental results under fully reversal uniaxial and torsional fatigue loading.
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Using a plain uniaxial fatigue S-N curve generated under a load ratio, \( R > -1 \), it is then possible to determine the mean stress sensitivity index value, \( m \), using the following expression:

\[
    m = \frac{\tau_a}{\sigma_{n,m}} \left( 2 \frac{\tau_a - \tau_{a,Ref}}{2\tau_a - \sigma_{n,m}} - \frac{\sigma_{n,\alpha}}{\tau_a} \right) \tag{Eq. 3-9}
\]

where \( \tau_a \), \( \sigma_{n,m} \) and \( \sigma_{n,\alpha} \) are stress components of the critical plane for the above plain uniaxial fatigue S-N curve. The value of the material constant ‘\( m \)’ can vary between zero and one.

In [34], a threshold value, \( \rho_{lim} \), is proposed for the effective value of the critical plane stress ratio whose value can be calculated as follows:

\[
    \rho_{lim} = \frac{\tau_A}{2\tau_A - \sigma_A} \tag{Eq. 3-10}
\]

In situations where \( \rho_{eff} \) is larger than \( \rho_{lim} \) the multiaxial fatigue assessment is proposed to be carried out taking \( \rho_{eff} = \rho_{lim} \), that is, when critical planes experience high values of \( \rho_{eff} \).

The methodology suggested in [34] for multiaxial fatigue assessment is summarised in Figure 3-8. It requires the linear-elastic stress distribution along the focus path to be known. The graph of maximum shear stress amplitude, \( \tau_a \), and \( \rho_{eff} \) versus distance, \( r \), from stress...
concentration apex, point O, can be obtained, as shown in Figure 3-8b. Using the corresponding modified Wohler curve estimated based on the calculated values of $\tau_a$ and $\rho_{eff}$, the number of cycles to failure at any point along the focus path can be easily determined using Eq. 3-11.

\[
N_f = N_A \left[ \frac{\tau_{A,ref}(\rho_{eff})}{\tau_A} \right]^{k_r(\rho_{eff})}
\]

**Eq. 3-11**

*Figure 3-8 Proposed procedure to apply the MWCM along with the PM to estimate the notch fatigue lifetime in the medium-cycle fatigue regime [34]*

Subsequently, using Eq. 3-4, it is possible to determine the critical distance value, $L_m$, for any value of $r$. The component under investigation is assumed to have reached fatigue failure under multiaxial loading if the following condition is achieved:

\[
\frac{L_m}{2} - r = 0 \Rightarrow \frac{AN_{f,e}^B}{2} - 1 = 0
\]

**Eq. 3-12**

The flow chart shown in Figure 3-9 summarises the procedure outlined in [34] for multiaxial fatigue assessment of notched components subjected to complex multiaxial fatigue loading.
It should be noted that the critical length versus number of cycles to failure, \( L_M \) vs. \( N_f \), must be calibrated by using results of fully reversed fatigue tests. In [34], the accuracy and reliability of the proposed multiaxial fatigue assessment methodology was checked through systematic experimental investigation as well as application of the approach to several sets of data obtained from the literature. The MWCM applied along with the TCD method was successful by yielding prediction within the CA scatter bands of the data used to calibrate the method itself [34].

In the following chapter, some of the experimental tests performed on riveted bridge members and connections obtained from the technical literature are presented. The main aim and principal findings of these experimental investigations are also summarised. Some of these case studies are adopted in the current thesis for fatigue assessment based on both the S-N approach and the TCD method.
4 Fatigue tests of riveted members and connections

The riveting technique was the main method used to connect structural steel elements until the early 20th century. Many tests have been performed to experimentally study the static and fatigue behaviour of riveted members and connections. A large portion of these tests was performed on small-scale samples specifically on riveted connections because of the simplicity of this type of test in terms of sample preparation and test execution [12]. Because of the large volume of test data, a more reliable statistical analysis was reached. The purpose of the majority of the small-scale experiments was to investigate the main factors affecting the fatigue performance of riveted connections and simple shear type riveted joints (e.g., clamping force and bearing ratio) [39–41]. Full-scale tests on riveted members of existing or demolished bridges and newly laboratory built riveted components have also been carried out but in limited numbers. These tests, while few, eliminated the need to make further assumptions due to size effects and provided realistic insight about the fatigue performance of such members and details. For instance, previous full-scale fatigue tests have shown that a high degree of inherent redundancy is present in riveted built-up sections capable of redistributing stresses from a cracked component of a built-up member to uncracked ones. This kind of behaviour has been shown to extend the fatigue life of full-scale riveted members; simple shear joints and small-scale specimens do not allow such behaviours to be investigated [42, 43]. It is obvious then that a more representative result in terms of fatigue performance of old riveted bridges can be expected when considering test data obtained from full-size riveted members from old riveted bridge structures. The advantage of experimental investigations on such elements is the ability to obtain and observe the effect of many parameters influencing fatigue performance, such as, rivet clamping force, material properties, corrosion damage, hole misalignment and other construction mechanical defects or a combination of them as compared to small-scale laboratory manufactured specimens. Finally, size effect can also play an important role on the obtained fatigue test results. This is due to a greater number of fatigue-prone defects such as rivets, rivet holes and other stress concentration sites from which fatigue crack initiation and development could take place in full-size riveted members. Hence it is not unlikely to expect a lower fatigue life for a full-size specimens compared to that of a similar small-size specimen [12].
In the following section, a brief description of full-scale tests on riveted bridge members published in various technical reports, journal articles and conference proceedings is presented, and the main findings of the experimental research are summarised.

4.1 Fatigue tests on riveted built-up girders

Reemsnyder, H. S. [47] considered strengthening methods of riveted members by investigating effects of replacing rivets with high-strength friction grip bolts in full-scale truss members. The primary aim was to examine the effect of rivet clamping force on the fatigue life of tested specimens. A total of eighteen riveted specimens (mostly newly-manufactured truss members and two recovered from an old ore bridge) were tested at stress ranges of 125 and 157 MPa with corresponding stress ratios, \( R \), equal to -0.56 and -0.65. In cases where rivets were replaced with bolts at locations of expected cracking, fatigue life increase by a factor ranging between 2 to 6 times was observed.

![Figure 4-1](image)

Figure 4-1. A detail of the truss joint and a cross-section of the truss member that was fatigue tested by Reemsnyder in 1975 [48]
Fatigue tests of riveted members and connections

Rabemanantsoa, H. and Hirt, M. A. [49] performed four-point bending fatigue tests on four mild steel rolled girders (HEB 1000). The girders were intended for temporary use in railway bridges and were in excellent condition (never been in service). All the girders had a cover plate riveted to the tension flanges and were subjected to relatively low stress ranges ranging between 78 to 90 MPa at a stress ratio, $R$, equal to 0.1. Fatigue cracks were observed to initiate from the rivet hole in the lower flange. Drilling a stop hole at the crack tip or addition of an extra cover plate in the lower tension flange was investigated as possible in-field reinforcing measures for crack arrest. The results showed up to 3 times longer fatigue life for reinforced girders, demonstrating that considerable increase in-service life can be achieved if the fatigue-cracked girder is reinforced in a suitable manner.

Out, J. M. M., Fisher, J. W. and Yen, B. T. [50] conducted CA fatigue tests on severely-corroded riveted built-up girders in four-point bending at stress ranges ranging between 56 and 75 MPa. Heavy corrosion damage in three of the girders resulted in crack initiation at the flange edge in the corroded zones rather than from the rivet hole. Effects of reduced temperature (-40°C) on fatigue and fracture characteristics of riveted girders were also investigated in these tests. A run-out result was obtained for the fourth girder (40 million cycles) which was also tested at -40°C with stress range level of 63 MPa. The test results showed that, in cases where corrosion damage had not severely reduced the plate thickness, fatigue cracks initiation is observed at the rivet holes.

Baker, K. A. and Kulak, G. L. [51] performed two sets of constant amplitude (CA) fatigue tests on full-scale specimens. In the first series, which was performed at a stress range of 216 MPa on six rolled wide-flange beams with punched holes in tension flanges, three beams were tested with empty holes and the other three had high strength friction bolts (HSFB) inserted in the holes. The primary aim was to investigate the effect of rivet clamping force on fatigue life of rolled girders with empty holes in tension flanges, in relation to fatigue strength of non-bearing riveted connections under cyclic tensile stress. Fatigue cracks in all six girders were developed at a rivet hole in the net section of the tension flange. The results showed up to 16 times longer fatigue life for girders with HSFBs as compared to flanges with empty rivet holes.
Fatigue tests of riveted members and connections

The second series of CA fatigue test results reported in [51] correspond to axially loaded truss-bridge hangers taken from a highway bridge and tested at stress range of 138 MPa with a stress ratio, \( R \), equal to 0.14. These specimens showed higher fatigue strength as compared to beams with empty holes in tension flanges.

Fisher, J. W., Yen, B. T., Wang, D., and Mann J, E. [45] reported results of four-point bending fatigue tests on thirteen full-scale riveted built-up I-girders obtained from three different old bridges. The tests were conducted at stress ranges of 83, 103 and 124 MPa where the stress ratio varied between 0.1 to 0.54. For heavily corroded girders (three of the tested specimens), due to corrosion-notch effects, fatigue crack development was observed in the gross section. For the remainder of the specimens cracking occurred at rivet holes in the tension-flange-to-web connections. The test results of seven girders tested at periodic intervals of reduced temperature (below -40°C) during the testing indicated insignificant effect on the fatigue and fracture characteristics of the components due to reduced temperature. Substantial level of structural redundancy was exhibited by the riveted built-up girders allowing for large cracks in one component to be sustained by redistributing load to other parts without any brittle failure.

Abe, H. [52] performed four-point bending fatigue tests on nine riveted built-up stringers obtained from an old railway bridge at stress ranges ranging between 73 and 137 MPa. The stringers contained high degrees of corrosion damage at the mid-span area where horizontal wind-bracing diagonals were attached to their lower flanges. The reported results confirmed that in all of the stringers with fatigue failure, crack initiation occurred at a rivet hole edge in the corroded region.

Bruhwiler, E., Smith, I. F. C., and Hirt, M. A. [41] reported the results of fatigue tests performed on three different types of riveted bridge girders. The first series was conducted on four rolled girders with riveted cover plates on tension flanges [49]. They also investigated the fatigue life of six built-up girders and three lattice girders obtained after demolition of two different road bridges built in 1884 and 1891 respectively. Both girder types were made of wrought-iron. The constant-amplitude fatigue tests were conducted in four-point bending at a stress ratio, \( R \), equal to 0.1 for all the girders with stress ranges varying between 50 to
120 MPa. The failure criterion was assumed as a 0.2 mm increase in girder deflection due to an observed fatigue crack. The study concluded that corrosion damage, unless present at the rivet hole region, does not affect the fatigue strength of the member. Comparison of the results with those published by Rabemanantsoa and Hirt [49] showed that wrought-iron riveted girders exhibit fatigue strength similar to that of early mild steel riveted members.

Mang, F. and Bucak, Ö. [53] carried out fatigue tests on a whole Blumberg bridge section as well as beams and components taken from the Blumberg and the Stahringen railway bridges built in 1887 and 1895, respectively. Constant amplitude fatigue tests were performed in three- and four-point bending at stress ranges between 52 and 186 MPa in which the stress ratio, $R$, varied between 0.1 and 0.3 for some of the tested specimens. Good agreement was seen when comparing the results with those of previous studies on full-scale riveted members [42, 44, 47, 50].

Åkesson, B. [46], in 1994, investigated the fatigue strength of mild steel riveted built-up stringers obtained from the Vindelälven Bridge built in 1896. The tests were carried out on nine of the stringers in four-point bending with relatively low to moderate stress ranges of 40, 60, 80 and 100 MPa. The stress ratio, $R$, for these full-scale tests varied between 0.14 to 0.28. At low stress ranges, stringers sustained up to 10 to 20 million load cycles. The fatigue cracks in most the cases originated from a tension flange rivet connection outside the mid-span region and only in one case initiated from a neck rivet hole in the mid-span region. The study concluded that a high degree of inherent structural redundancy was present in the tested built-up I-beams. It was observed that the stringers sustained a large number of stress cycles before allowing propagation of a fatigue crack from one component of the girder to another.

Adamson, D. E. and Kulak, G. L. [55], in 1995, reported the results of a series of full-scale fatigue tests carried out on six mild steel riveted built-up stringers obtained after dismantling a truss railway bridge built in 1911 near Alberta. Stringers contained some mild corrosion damage at the location of the horizontal gusset plate connections to the underneath of the bottom flanges used to attach the lateral bracing bars between two adjacent stringers. Tests were performed in four-point bending with stress ranges of 62, 66, 68 and 73 MPa. Fatigue failure criterion was defined as the development of a fatigue crack through one component.
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of the built-up stringer and detection of a new crack in another component. In majority of the fatigue failure cases, cracks originated from the rivet hole in the gusset plate connection in the mid-span region of the stringers.

Dibattista, J. D. and Kulak, G. L. [56], in 1995, reported the results of fatigue tests on seven full-scale tension diagonals removed from the same railway bridge described above in the study by Adamson and Kulak [55]. Each specimen consisted of half a truss diagonal with the corresponding end gusset plate connection. Specimens were in a relatively good condition with only very light corrosion damage. The specimens were loaded in cyclic uniaxial tension with relatively low net section stress ranges of between 58 to 73 MPa and stress ratios, $R$, equal to 0.12 or 0.15. The fatigue failure criterion was set similar to that in [55] where failure was taken as the development of a fatigue crack through one component of the built-up section and detection of a new crack in another element. The results of the experiments showed that lower fatigue strength is observed for the connections in which rivets are subjected to bearing pressure.

Helmerich, R., Brandes, K. and Herter, K. [57] published results of three sets of fatigue tests on riveted bridge members. The first series of tests were performed on two mild steel truss girders obtained from a suburban train viaduct built in 1902. In the second series, four-point bending fatigue tests were conducted on three riveted built-up girders made of mild steel obtained from a road bridge in Berlin built in 1906. The final series of tests were carried out on four riveted wrought-iron cross-girders taken from an old suburban railway bridge constructed in 1890. The cyclic stress range used in the tests varied between 85 and 135 MPa with a stress ratio of $R = 0.1$. In all test series, fatigue failure was taken as an increase of member deflection of more than 0.2 mm or once a whole section of an element has been severed by a propagated crack. In some cases, tests were continued until crack propagation into the web plate was also observed. Most of the fatigue cracks initiated from a rivet hole. The study concluded that the fatigue behaviour of wrought-iron members in rolling direction during standard tests is comparable with that of early mild steel.

Kadir, Z. A. [58], in 1997, performed another series of tests on three of the Vindelälven Bridge built-up stringers at 100 and 97 MPa stress range levels. The results were seen to conform
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with the results obtained in full-scale tests by Åkesson [46] (presented above). The stress ratio, $R$, was equal to 0.28 for two of the stringers and 0.16 for the third one.

Al-Emrani, M. [7, 54], published results of two series of constant amplitude fatigue tests on eight built-up stringers of the railway bridge over the Vindelälven river (continuing the experiments performed by Åkesson [46] and Kadir [58]). The first series of fatigue tests were designed to assess the existence of a constant amplitude fatigue limit (CAFL) for old steel riveted bridge members, that is, a stress range at or below which fatigue failure is assumed to not occur even for an infinitely large number of cycle. The stringers were tested at stress ranges of 40 and 60 MPa initially up to 10 to 20 million stress cycles and consequently at a much higher stress range (in the order of 100 MPa) to complete failure. The test results were then compared to the results of the previous tests on the Vindelälven Bridge performed by Åkesson [46] and Kadir [58]. It was concluded that the CAFL given by the Eurocodes detail category 71 (CAFL = 52 MPa) [27] is applicable for old steel riveted details. The second series of fatigue tests were carried out on three of the Vindelälven Bridge built-up stringers at stress range levels of 60 to 100 MPa [12]. These tests were performed to assess the efficiency of drilling stop holes as a temporary arrest for propagating fatigue cracks. The technique was found to be effective in arresting the crack development by delaying the propagation by up to 215,000 loading cycles.

Bassetti, A., Liechti, P. and Nussbaumer, A. [60], in 1999, performed constant-amplitude fatigue tests on four mild steel cross-girders obtained from a railway bridge constructed in 1901 over the Hinterrhein River in Switzerland. The built-up I-shape cross-girders had a riveted cover plate on the compression flange. The girders were tapered at either side with both ends having shorter web height compared to the central segment. The girders were tested in four-point bending with stress ranges of 72, 80 and 91 MPa at a stress ratio, $R$, equal to 0.1. Fatigue cracking occurred exclusively at rivet holes in the tension flange and in majority of the cases, within the mid-span constant-moment region. The experiment also investigated the effectiveness of various conventional repair techniques such as replacing rivets with high-strength friction bolts in the cracked hole and addition of a cover plate patch over the cracked hole. It also considered bonding pretensioned strips of carbon fibre reinforced plastic (CFRP) to the cracked section as a new repair method. The results concluded that all the conventional
repair approaches, even though successful in case of small cracks, but are more local and hence highlighting the importance of fatigue crack detection at an early stage. However, CFRP laminates can be used as a repair technique as well as a preventative measure applied to a larger section of a bridge member and hence to increase fatigue strength of all the rivets covered.

**Xie, M., Bessant, G. T., Chapman, J. C. and Hobbs, R. E.** [29], in 2001, published the results of a series of fatigue tests on three rolled cross-girders with a riveted cover plate on the top and bottom flanges taken from a demolished bridge after 120 years of service life. As the average thickness of the original cover plate was reduced in some locations by nearly 50% due to corrosion damage, an additional 254x15 mm cover plate was added to the top flange of all three girders. The first cross-girder was tested at an average stress range of 87 MPa. The applied constant-amplitude stress range in case of the second and the third girder was 122 and 110 MPa, respectively. All the tests were performed in four-point bending with stress ratio equal to 0.1. Fatigue crack initiation for the first and second girders occurred at a rivet hole near the mid-span in the constant-moment region. In case of the third girder, a horizontal crack emanated in the web plate near the neutral axis in the shear region of the girder. It was found that corrosion damage had reduced the average web plate thickness by 50-80% in this region. The study also investigated tensile fatigue resistance of some of the rivets as well as parts of the bottom cover plates recovered from the cross-girders.

**Greiner, R. and Matar, E. B.** [61] published a paper in 2006 that presented the results of a fatigue test series on mild steel stringers obtained from two railway bridges in Austria after demolition. The first built-up stringer type was taken from a bridge built in 1903 and consisted of four flange angles, two filler plates in top flange and a web plate. The second type of built-up stringers was salvaged from a bridge constructed in 1913 and comprised of four flange angles, a web plate and unsymmetrical riveted cover plates in the top and bottom flanges. All the stringers were in fairly good condition with no severe corrosion damage. Two of each stringer type were tested in three-point bending at a stress range of 120 MPa with stress ratios varying between 0.08 to 0.217. The tested specimens were half the full length of the original stringers and either the central half or the end half. Two fatigue failure and two run-
out results were obtained in these tests. All the fatigue cracks were initiated at a rivet hole near the maximum moment region.

Pipinato, A., Pellegrino, C., Bursi, O. S. and Modena, C. [62], in 2009, experimentally investigated the fatigue behaviour of mild steel riveted shear diaphragms removed from a railway bridge built in 1918 in Italy. The bridge was a twin-girder structure with each rail directly carried by the short shear diaphragms connecting the two adjacent built-up girders. The cyclic shear stress range used in these fatigue tests varied between 264 and 353 MPa with stress ratio, $R$, between 0.1 and 0.14. Fatigue failure in all four tests occurred exclusively in the rivets carrying shear and resulted in the heads popping off. Once a shear rivet failed, the other rivets also failed rapidly due to increased shear stresses. The shear fatigue resistance of riveted shear details appeared to be higher than that predicted by the design codes (Eurocodes detail Category 71, for non-preloaded bolts in shear). It was argued in this study that higher shear fatigue strength of riveted shear details could be partly related to lack of thread roots and thread run-outs in rivets as compared to bolts.

Fatigue damage mechanism in most of the case studies mentioned above followed a very similar pattern with fatigue crack initiating at a rivet hole in locations experiencing highest tension forces. It was observed that the loss of rivet clamping force during the service life of the member could significantly reduce the fatigue life especially in connections where rivets are subjected to bearing pressures. The opposite was true in cases where rivets were replaced with high-strength friction bolt resulting in substantial increase in the remaining fatigue life of the girder. However, some studies suggested that this remedy is more effective if carried out on small cracks which highlights the importance of inspection for fatigue crack detection at early stages. Sub-zero temperatures were noticed no have no major impact on the fatigue and fracture characteristics of riveted girders. Corrosion damage, on the other hand, was seen to affect the location of fatigue crack initiation only in cases where there is considerable reduction in cross-section of the plate in tension or shear zone. A high degree of inherent structural redundancy was exhibited by the built-up girders allowing large number of loading cycles to be sustained before propagation of fatigue crack to adjacent components. In general, wrought-iron and mild steel riveted girders exhibited comparable fatigue strength.
4.2 Fatigue tests on double-angel riveted connections

Double angle connections are usually used to join the primary load-carrying members of metallic bridges, that is, stringer-to-floor-beam connections. These connections are typically considered and idealised as pinned and hence designed to only transfer stringer-end shear forces to the floor-beam web. However, in reality, some bending moment is also developed by these double angle connections due to a certain amount of inherent end rotational stiffness. Since double angle connections could be subjected to large repetitions of loading cycles during their service life, these unforeseen secondary flexural stresses can cause fatigue damage if the constant amplitude fatigue limit is exceeded. Evaluation of the fatigue performance of these connection types in riveted railway bridges therefore becomes of great importance due to, for instance, reliability and safety issues. Despite the fact that experience has shown that stringer-to-floor-beam connections are prone to fatigue damage [7, 10, 59, 60, 61], the amount of experimental and theoretical investigations to evaluate their fatigue performance is relatively low. In this section, small- and full-scale experimental investigations on fatigue performance of stringer-to-floor-beam connections found in the technical literature are summarised and the main findings obtained from these studies discussed.

Wilson, W. M. and Coombe, J. V. [63], in 1939, published the results of one of the first series of experiments with respect to fatigue performance of stringer-to-floor-beam connections. The constant amplitude fatigue tests were performed on three series of T-connections (C1, C2 and C3) representing a short length of a stringer-to-floor-beam connection. All the specimens were identical in the fact that each comprised of four angles, four filler plates, two central plates (representing the stringer web) and a spacer plate (representing the floor-beam web) fastened together using 25 mm diameter rivets. The C1 and C2 specimen types had 4 rivets acting in tension with the shorter leg of the angle attached to the floor-beam web. The only difference between these two specimen types was the angle thickness. As for C3 specimens, six tension rivets were used with the angles having different dimensions and the longer angle leg being connected to the floor-beam web. Detail of the test specimen C1, C2 and C3 is shown in Figure 4-3. A total of nine specimens (three of each specimen type) were tested in axial tension with a stress ratio of \( R = 0.0 \). The model shown in Figure 4-2 was used by the authors (Wilson model) to calculate the flexural strength of the angles. Wilson model assumes that the outstanding leg of the angle is fixed both at the rivet centre line and at the
angle fillet and the point of contra-flexure (i.e., the point at which bending moment changes its sign from negative to positive) is located at the mid-length between the two fixed points.

![Diagram of riveted member with labels: Stringer web, Point of contra-flexure, Floor-beam web, b/2, b/2, g]

*Figure 4-2 The fixed-end beam model suggested by Wilson and Coombe [63]*

Specimens C1 and C2 were designed initially to fail in the tension rivets and specimens C3 were intended to develop fatigue failure in the outstanding leg of the connection angle. The C2 type specimens had fatigue cracking exclusively in tension rivets, however, C1 had one failure in the outstanding leg of the angle, one in a tension rivet and one run-out result. The fatigue strength (defined as the maximum stress causing fatigue failure at $2 \times 10^6$ cycles) of the rivets in tension for C1 and C2 specimens was estimated to be about 112 and 140 MPa, respectively. The average unit tension was calculated by dividing the maximum applied load to the total area of rivets in tension. As for C3, two fatigue failures in the outstanding leg of the angle and a run-out result was obtained resulting in an average fatigue strength of 350 MPa in flexure to be estimated for the outstanding leg of the angle in this specimen type. Since this value is greater than the yield stress of steel used to fabricate the test specimens, it was concluded that the Wilson model overestimates the flexural stresses in the outstanding leg of the angle. However, the behaviour of the angle was not greatly different from that suggested in the Wilson model. In another study [66], Wilson proposed the following empirical equation to be used for the design of double angle connections to determine the gauge distance in order to avoid fatigue damage and cracking:

$$ g \geq \left(\frac{Lt}{8}\right)^{1/2} \quad Eq. 4-1 $$
where ‘g’ is the gauge distance (see Figure 4-2), ‘L’ is the span of the stringer and ‘t’ is the thickness of the outstanding leg of the angle.

**Figure 4-3** Details of specimens used by Wilson and Coombe [63]
Wang, D. [64], in 1990, experimentally investigated the fatigue behaviour and load-deformation relationship for mechanically fastened double angle shear connections representing stringer-to-floor-beam connections in steel bridges. Sixteen T-connections were cut out from I-shape built-up steel beams taken from a railway bridge. The top flange section of the beams near the ends with no cover plate were used to prepare the specimens. The existing connection between the angles (152 x 152 x 15.9) and the web consisted of three staggered rivets 25 mm in diameter (on the angle leg adjacent to the beam web). Tests on specimens were performed in pairs. For that purpose, a 9.5 mm thick spacer plate, simulating the floor-beam web, was used between the two geometrically identical specimens. The outstanding legs of the identical specimens and the spacer plate were joined using a single column of three high-strength bolts (25 mm diameter) as shown in Figure 4-4. The bolts were tightened to develop clamping stresses of 136, 226 and 317.8 MPa in different specimens.

![Diagram of T-connections tested by Wang]

In addition to the variable clamping stress, the prepared test specimens had four different gauge distances, \( g \), of 52, 71, 90 and 110 mm. Fourteen pairs of specimens were prepared.
and tested under constant amplitude axial fatigue loading with calculated flexural stresses in the outstanding leg of the angles varying between 79.3 and 442.9 MPa. The flexural stresses were determined using the fixed-end beam model proposed by Wilson [66] mentioned above. The stress ratio, $R$, in these tests was between 0.23 to 0.6. Fatigue cracks in the majority of the cases initiated at the angle fillet. Only in one case after the initial fatigue crack started at the fillet, a second crack also formed in the outstanding leg of the angle at the bolt-line. Four of the paired specimens which were tested at relatively low stress levels and did not sustain any fatigue failure, were consequently tested at higher stress levels and the cycle count was restarted from zero. The specimens were always tested as pairs but fatigue cracking usually occurred only in one of the specimens. After the first fatigue crack initiation, the applied load level was adjusted manually to maintain a relatively constant deflection. In such cases, the uncracked specimens with identical specifications were joined together and tested at a slightly reduced applied load. It was observed from the test results that the level of stresses in the bolt shanks were much lower than that calculated based on common engineering practice (dividing applied load by the total area of rivets in tension) which shows the effect of clamping force. Strain measurements on the outstanding leg of the angle also showed higher flexural stresses near the fillet compared to stress levels near the bolt line. This confirms that the point of contra-flexure is not located at the mid-length between the fillet and the bolt line as assumed by Wilson model [63]. The study also concluded that the most effective way of reducing the fatigue damage susceptibility of double-angle shear connections is to increase the flexibility of these connections (reduce end rotation) by optimising the influential geometric parameters, that is, angle thickness ($t$) and gauge distance ($g$). It was noted that increase of the clamping force does not significantly modify the connection flexibility and while being locally beneficial by boosting the fatigue performance at the fasteners region, it also reduces the fastener stress variation.

Abouelmaaty, W., Maragakis, E., Itani, A. and Douglas, B. [65], in 1997, performed full-scale constant-amplitude fatigue tests on two newly-manufactured stringer-to-floor-beam connections similar to those used in a railroad bridge near Nevada. Each test specimen consisted of a 3-meter-long hot-rolled steel stringer and a 1.22-meter-long built-up floor-beam connected together using two $L152 \times 152 \times 13$ mm angle cleats. The angles were attached to the stringer and the floor-beam web using 22 mm diameter bolts (due to
unavailability of rivets). A hinge support was used at the other end of the stringer to allow calculation of the fully-fixed end-moment percentage transferred through the double-angle connection. The specimens were subjected to reversal load of ± 196 kN.

Figure 4-5 shows the set-up and the loading condition used in these tests. Fatigue cracks in both test specimens initiated in the floor-beam web at the location opposite the stringer’s top and bottom flanges. These cracks were created due to punching contact of the stringer flanges to the floor-beam web during reversal loading. In real cases, since the loading cycle is from tension-to-tension, these types of crack would only appear at the stringer’s bottom flange contact area. Another reason for fatigue crack initiation in the floor-beam web could be the out-of-plane displacement of the floor-beam web under the applied load and due to torsional flexibility. Further damage due to elongation was observed in two of the bolts connecting the angle to the floor-beam web. Additional cracks were also initiated in the stringer web near the angle leg in the final stages of the tests. The amount of bending moment carried by the double angle connection was estimated to be about 8.5% of the stringer’s fixed end-moment.

![Figure 4-5](image)

*Figure 4-5 Full-scale stringer-to-floor-beam connections tested at the University of Nevada [65]*
Fatigue tests of riveted members and connections

Al-Emrani, M. [12], in 2002, published the results of a series of full-scale experiments investigating the fatigue performance of riveted stringer-to-floor-beam connections. The objective of this study was to assess the effects of secondary-induced stresses on fatigue crack initiation and propagation in such connections. Three full-scale bridge ‘parts’ (specimen I, II and III) which were taken from the same span of the Vindelälven Railway Bridge were used in this experiment. Each test specimen consisted of four longitudinal stringers attached to three transverse floor beams, as shown in Figure 4-6 (a). The webs of the stringers and the floor-beams were joined by means of riveted L-profile double-angle connections (L100 × 75 × 9). The test specimens also had the sway- and cross-bracing elements of the original bridge attached to them, as well as the remains of the tension chord wind-bracings. All three test specimens were in relatively good condition with only slight corrosion spotted near the connection between sway- and cross-bracing elements and the stringers. The stringer-to-floor-beam connections appeared to be assembled on site using field-driven riveting process to attach the connection angle to the floor-beam web. Punching method had been used to form the holes in the connection angles. Pre-existing cracks were found near the fillet of the connection angles at the level of the upper rivet for all test specimens which indicated that the connection angles were highly stressed during the service life of the bridge.

Specimens were set up in the testing machine with a simple support positioned under each floor-beam end at the location where stringers meet the floor-beam. Constant amplitude fatigue tests were performed with all the stringers in the test specimens being simultaneously subjected to four-point bending. A load range of $P = 100$ kN was applied at the centre line of the stringers in all three tests with the corresponding minimum and maximum applied loads equal to 80 and 180 kN. Loading was continued up to 10 million cycles in order to investigate fatigue crack propagation in the connection angles. Static tests were also carried out both before and during the fatigue tests in order to assess the load-deformation characteristics of the specimens and their rotational stiffness.

The fatigue damage mechanism for all three test specimens followed very similar pattern. Initially, a new fatigue crack emanated near the fillet of the connection angles at the level of upper rivets as a result of out-of-plane distortion (see Figure 4-7 (1)).
Figure 4-6 (a) Test set up and overall dimensions of each test specimen (b) Detail of the stringer-to-floor-beam connection [67]

After the first crack propagated long enough to adequately reduce the local flexural stiffness of the outstanding leg of the angle at the level of the top rivet, a second fatigue crack initiated near the same location but at the level of the second top rivet row (see Figure 4-7 (2)). This was due to the fact that this location was the next stiffest part in the connection and started to experience higher proportion of the stresses imposed on the connection. Vertical propagation and joining of both cracks and development of the first crack through the entire thickness of the angle resulted in a considerable reduction in rotational stiffness of the angle at this stage and eventually a condition of self-arrest was reached (see Figure 4-7 (3) and (4)). At this point, the cracked section length in the outstanding leg of the angle near the fillet reached to about 40% of the total angle depth.
Another fatigue damage mode observed in all three of the test specimens was failure of the tension rivets connecting the outstanding leg of the connection angle to the floor-beam web.

![Crack length versus number of cycles](image)

**Figure 4-7 Fatigue crack propagation in stringer-to-floor-beam connections tested by Al-Emrani near the fillet of the outstanding leg** [67]

This type of fatigue failure was caused by the bending of the rivets due to flexure of the angle leg as well as the high stress concentration as a result of abrupt change in geometry at the rivet-shank-to-head junction. Rivet heads in the first and/or second upper rivet rows were observed to fracture and pop off in all three specimens at very early stages (around 0.5 to 0.9 million cycles). Inspection of the rivet fracture surfaces revealed pre-existing cracks in these rivets prior to fatigue testing. It was also concluded that rivet head failure was a major contributing factor in reducing crack propagation rates in angles and causing a total arrest condition in the angle fillet fatigue cracks. It occurred due to the release of most of the stringer-end restrained deformation and the associated stresses acting on the connection. Some of the other findings of the fatigue tests on the stringer-to-floor-beam connections are summarised below:

- Rivet clamping was found to have minor influence on the stiffness of the connection and the magnitude of the stresses at the fillet was virtually unaffected by the magnitude of rivet clamping force.
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- Bending stress distribution down the height of the fillet of the outstanding leg of the angle was seen to be non-linear having higher stresses at the level of rivet rows. This location appears to be stiffer and hence attracting larger percentage of the forces imposed on the connection.

- Imperfections, such as gaps between the back of the outstanding leg of the connection angles and the floor-beam web, could adversely affect the fatigue performance of double angle connections. It was observed in the static and fatigue tests on the that one of the specimens containing such a gap, displayed slightly varied load-displacement characteristics and fatigue behaviour compared to other two specimens.

- Despite a vertical fatigue crack covering up to 40% of the angle height and additional fatigue damage sustained in the tension rivets, the load-carrying capacity of the stringer-to-floor-beam connections were unaffected. The rivets with popped-off heads were seen to still contribute to the shear load capacity of the connection.

- The stringer-to-floor-beam ‘shear’ connections had rotational stiffness capable of developing stringer-end moments as large as 67% of a fully-continuous beam.

4.3 Concluding remarks

The experimental investigations previously carried out on full- and small-scale riveted members and connections of metallic bridges were presented in this chapter. The fatigue-critical details and fatigue damage mechanisms in such structural details were identified.

The literature review revealed double-angle riveted connections as one of the fatigue-critical details encountered in railway bridges. These connection types are subjected to unforeseen secondary deformation-induced flexural stresses during their service life. Therefore, accurate fatigue life evaluation of such connection becomes of great importance because of reliability, safety and economical factors. In the following chapters, to assess the predictive capability of the TCD and the S-N methods, fatigue assessment of some of the details presented in the literature review is carried out. This allows for comparison of the fatigue life estimated based on the S-N and the TCD methods. Initially, an FE benchmark study is performed to confidently identify the modelling techniques and mesh refinement required for such fatigue assessment.
5 A benchmark study on finite element modelling for fatigue assessment

5.1 Introduction

As mentioned earlier, when applying the Theory of Critical Distances (TCD) for fatigue life assessment, the linear-elastic stress field ahead of the stress concentration should be determined to calculate the average reference stress value (see Figure 2-4). One of the most convenient ways to do so is by using finite element (FE) modelling. In the last few decades, considerable development has occurred in the field of FE modelling which allows easier and accurate determination of the linear-elastic stress fields in complex three-dimensional (3-D) geometries and in the presence of complex loading conditions. However, accuracy of the FE analysis results can be highly sensitive to the modelling technique as the obtained stresses from the FE model are often near the location of stress singularity.

This chapter intends to verify the reliability of the FE modelling techniques which will later be used to model and analyse refined 3-D models of various simple and complex fatigue-prone bridge details made of old structural metals. The aim is to identify the element type(s) and optimum mesh density that is efficient (in terms of computational cost) and sufficiently accurate in calculation of the linear stress distribution ahead of the stress concentration feature. The above element type(s) and sizes should allow for correct determination of stress field near a stress raiser in all directions as well as through the thickness for any given detail. It becomes vital especially in the case of more complex fatigue-prone details subjected to complex loading conditions such as stringer-to-floor-beam connections in bridge structures.

Before attempting to develop such complicated models, as a benchmark study, a 3-D steel lap joint with a single rivet is modelled and analysed. This elastic model incorporates friction between connecting parts, contact features, clamping force and other geometrical characteristics. Since contact interactions exist between surfaces of different members of the lap joint, despite assuming linear-elastic behaviour for the material, the overall global problem is non-linear, and hence, incremental loading is applied to investigate the elastic stress concentration factor.

The reason a single lap joint (SLJ) was selected for the purpose FE modelling verification and parametric benchmark study is the existence of secondary bending stresses generated in different components of these connections due to eccentric loading of the plates. This is very similar to the bending stresses occurring in stringer-to-floor-beam connections because of
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out-of-plane deformation of the angle leg, and causing fatigue damage. In SLJs, eccentric plate loading generates shear forces which are transferred from the upper to the lower plate through the plate-rivet contact resulting in the rotation of the rivet. Another outcome of this rivet rotation is the uneven bearing of the rivet shank over the thickness of the hole and the consequent bending of the plates around the rivet head. This plate bending combined with rivet tilting generates higher stress concentration factors (SCF) at the hole region. The stress gradients through the plate thickness are uneven, which shifts the location of these peak stresses (maximum tensile and compressive stresses) to the inner surface of plates where high contact pressures exist [68]. SLJs, because of their relative simplicity, have also been more widely investigated and, therefore, their behaviour is well understood, which makes them suitable for benchmark studies and verification purposes.

The element type(s) and optimum mesh density combination in the FE model that are capable of successfully capturing the behaviour of SLJs could be considered for the FE modelling of simple and more complex riveted details in the remainder of this study for fatigue life predictions. The findings could also assist in creating a set of ‘universal rules’ that can be followed when performing fatigue assessment of similar details using the FE method.

5.2 Parametric study of FE mesh for a single lap joint model

The 3-D FE model of the SLJ, shown in Figure 5-1, was produced with the commercial FE-package Abaqus/CAE v6.10 [39]. The FE model consisted of two partially overlapping 10 mm thick steel plates fastened by a standard rivet with a round exposed head on either end. This connection can be considered as one unit of a multiriveted, single rivet-row lap joint which extends in both negative and positive 2-directions (see Figure 5-1). Linear-elastic material behaviour was assumed for the steel material with Young’s modulus and Poisson’s ratio values of 200 GPa and 0.3, respectively [69].

The end face A-B of the lower plate (LP) was constrained against translation along the 1-, 2- and 3-directions. The corresponding face C-D of the upper plate (UP) was restrained against translation along only 3-direction and subjected to a nominal tensile stress of 20 MPa. The faces A-C and B-D on the 1-3 plane belonging to both UP and LP are symmetry planes (since SLJ is assumed to extend in both the negative and positive 2-direction in a symmetric fashion) and therefore were assumed to have zero displacement in the direction normal to this plane.
(2-direction). This boundary and loading condition was selected to allow a symmetric condition to be created with identical stress fields to be expected in the upper and lower plates.

Several contact surfaces were modelled between the contacting bodies using the standard Surface-to-Surface contact method in Abaqus with finite sliding and master-slave surface algorithm [39]. A total of five contact pair surfaces were created (one plate-to-plate, two rivet head-to-plate and two rivet shank-to-hole surfaces) considering ‘Penalty’ friction formulation for tangential behaviour and the default ‘Hard Contact’ for normal behaviour.

The rivet shank was assumed to fully fit the hole space with ‘zero’ clearance considered (representing hot-driven rivets) [7, 10]. To model a 100 MPa clamping stress in the rivet, the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus/CAE v6.10 [39] was used, and a force
was applied across an internal cross-section surface at mid-length of the rivet on the 1-2 planes.

To ascertain the suitable brick element type between the 3-D 8-noded or 20-noded elements and identify the optimum mesh size for the FE analysis, a parametric study was performed. This was to deduce which element type and mesh density combination is capable of adequately capturing the stress variation through the plate thickness in SLJ specimens subjected to longitudinal uniaxial tensile loading.

A total of eight FE mesh types and densities were created. Five of these models were meshed using full-integration 8-noded brick elements (C3D8) having 2, 3, 4, 6 or 8 elements through the plate thickness with 24, 24, 28, 36 or 40 elements being used around the perimeter of the rivet hole, respectively (shown as 2t-24P, 3t-24P, 4t-28P, 6t-36P and 8t-40P in Figure 5-4). For the sake of comparison, four models of the SLJ were also created using the reduced- or full integration 20-noded elements (C3D20R or C3D20) with the corresponding FE mesh density of 4t-24P, 2t-24P, 4t-24P, 5t-32P, respectively, through the plate thickness and around the hole perimeter.

*Figure 5-2 Typical FE mesh used in parametric study. Six circular partitions (at 1 mm spacing) and 24 elements were used around the hole and rivet perimeter with four layers of elements through the thickness of each plate.*

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Figure 5-3 Principal stress variation through the upper plate thickness for models with fully- or reduced integration 20-noded brick elements and different mesh densities ('t' refers to the number of elements through the plate thickness, and 'P' refers to the number of elements used around the hole perimeter).

Figure 5-4 Principal stress variation through the upper plate thickness for models with full-integration 8-noded brick elements and different mesh densities ('t' refers to the number of elements through the plate thickness, and 'P' refers to the number of elements used around the hole perimeter).
Figure 5-5  Displaced shape and stress distribution in the SLJ under 20 MPa applied pressure for 100 MPa rivet clamping force (50× magnified). The symmetric stress fields in the UP and LP can be seen by comparing the shaded regions.

In Figures 5-3 and 5-4, the longitudinal stresses ($\sigma_{11}$) through the thickness of the upper plate (on the 2-3 plane for a 0° angular direction (see Figure 5-6) at the edge of the rivet hole) obtained for all investigated mesh densities are plotted. Figure 5-3 presents the stress gradient through the plate thickness for FE meshes with 20-noded brick elements while Figure 5-4 refers to the corresponding stresses for FE mesh densities with 8-noded brick elements. Comparison of Figure 5-3 with Figure 5-4 makes it evident that the stress gradient determined by 8-noded and 20-noded brick elements are not very similar especially in the outer surface region of the plate. It appears that the 8-noded brick elements, regardless of the degree of FE mesh refinement (number of ‘t’ or ‘P’), are unable to capture the effects of the secondary-
induced stresses due to bending of the plate around the rivet head, which is expected to generate compressive stresses (negative values, see Figure 5-3) in the outer side of the plates [68]. Moreover, the magnitude of the maximum tensile stresses determined at the inner surface of the plate are about 5% larger when 20-noded brick elements are used. One might argue that the dissimilarities in the magnitude of these peak tensile stresses are not very significant. Nevertheless, it should be borne in mind that even slight changes in the peak stress values near the stress concentration zone can result in substantial deviation of resultant estimated fatigue life, $N_f$. This is because, in order to estimate the $N_f$ value, the stress in the S-N relationship (see Eq. 3-1 and 3-2) is raised to the power of the inverse slope of the S-N curve, $m$.

Furthermore, the Abaqus/CAE v6.10 user’s manual [39] suggests to avoid using reduced integration 3-D 8-noded brick elements (C3D8R) in stress/displacement analyses since ‘hourglassing’ could occur resulting in uncontrolled mesh distortion. Hourglassing may cause the calculated strains at the only integration point in C3D8R elements to be all zero leading to inaccurate analysis results. The Abaqus user’s manual also adds that full-integration 8-noded brick elements (C3D8), when subjected to bending stresses, could suffer from the ‘shear locking’ numerical problem, by which shear strains (also known as the parasitic shear) that do not normally exist are created in these elements, which can invalidate the analysis output data. The above statements may explain the differences obtained in the diagrams in Figures 5-3 and 5-4.

On the contrary, reduced- or full-integration 20-noded brick elements (C3D20R and C3D20 respectively) seem to be successful in capturing an accurate stress gradient through the plate thickness as seen in Figure 5-3. The peak compressive and tensile stresses at the outer and inner surfaces of the plates, respectively, determined using C3D20R and C3D20 FE meshes, have an insignificant variation of less than 1%.

After all considerations, by careful assessment of Figures 5-3 and 5-4, it was concluded that C3D20R or C3D20 elements with 4 layers of elements through the plate thickness and at least 24 elements around the rivet/ rivet hole perimeter are sufficient to accurately establish a smooth stress state at the vicinity of the stress concentrations in riveted details subjected to bending stresses.

It should also be noted that in this parametric study, notch tip stresses are considered; hence, further mesh refinement is not expected to produce full convergence because of stress
singularities associated with stress concentrations. However, the TCD method, when applied to fatigue assessment, is shown to overcome this problem [4], because it does not consider stresses at points of stress singularities (single point) but takes into account the stress distributions within finite areas/volumes for fatigue life estimation purposes.

In the following chapters, the results of the benchmark FE analysis performed in this chapter are adopted in the finite element modelling of the fatigue-prone details for fatigue life assessment based on the TCD and the S-N methods. The next chapter investigates the accuracy of the TCD and the S-N method when used for fatigue evaluation of simple details, such as plates with single or multiple hole(s) and riveted single, double and butt joints. Comparison of the results of such investigation can help verify the applicability of the TCD method when applied to fatigue analysis of riveted details.
6 Fatigue life evaluation of simple details

Fatigue life evaluation of simple structural wrought-iron and mild steel details commonly used in old bridges is presented in this section. This chapter intends to compare and quantify the equivalence between fatigue life predicted by the S-N and the TCD methods. The TCD is applied in the form of the Point (PM), Area (AM), Line (LM) and the Volume method (VM). This allows for the validation of the applicability of different formalisations of the TCD in fatigue evaluation of simple details. Seven published experimental investigations have been obtained from technical literature and were used as the bases of the comparisons. Fatigue life estimation for the selected details was carried out according to the TCD method as well as the S-N approach. Finally, the estimated results were compared with the original experimental data to understand the potential of using of the TCD method.

6.1 Finite element modelling rules and recommendations for the TCD method

Fatigue life evaluation based on different formalisations of the TCD requires the linear-elastic stress field in the vicinity of the fatigue-critical location to be accurately calculated. For riveted bridge details, such fatigue-critical locations can be considered at the edge of rivet holes, the rivet head-to-shank intersection and the angle fillet. The advent of FE methods in the recent decades allows easier determination of the linear-elastic stress fields in complex 3-D geometries. Therefore, throughout this thesis, the commercial FE-package Abaqus/CAE v6.10 [39] is used for modelling and analysis of the investigated details and connections. With regard to the use of the FE method for fatigue evaluation of the bridge details using the TCD, the following recommendations have been followed:

- Following the findings of the benchmark study in the previous chapter, at least four elements through the thickness and 24 elements around the perimeter of holes or rivet shanks are used. This is to produce an accurate stress distribution and avoid underprediction of the results near the stress raiser.
- The parametric study performed previously showed that only fully and reduced-integration brick elements (C3D20 and C3D20R) are capable of reliably capturing the behaviour of members when subjected to bending stresses. Using these element types helps in capturing high stress gradients near the fatigue-critical locations in the case of more complex details. Another advantage of the brick elements is the possibility of
obtaining stress in the midside nodes. This could create a higher number of stress readout points and hence, a more accurate average reference stress calculation for the TCD fatigue assessment.

- A general FE modelling rule is also to keep the aspect ratio (i.e., the ratio of the longest to the shortest dimension) of FE elements below 1:3 to avoid inaccuracy in the results or unexpected modelling errors [10].

- Considering that the critical length value, \( L \), in the high-cycle region is determined to be around 1.7 mm, it is suggested to use a partitioning technique to create adequate stress readout points across the width and through the depth of the component in the vicinity of the stress concentration used for the processing required by the TCD method. This also allows for the stress transition in the region close to the stress raiser apex to be more gradual and smooth, which is beneficial when different formalisations of the TCD are adopted for fatigue analysis.

6.2 River Cree Viaduct – Flat wrought-iron specimens with one central circular hole

6.2.1 Experimental data

Burdon [70] carried out static and constant amplitude (CA) fatigue tests on small-size wrought-iron specimens and full-scale girders obtained from the River Cree Viaduct (Scotland) after 100 years of service life. Small-scale fatigue tests were performed at a stress ratio, \( R \), equal to zero on plain and notched flat specimens containing a newly drilled central hole designed to measure strength reduction caused by such stress raisers in this material. The investigated wrought-iron material without machining the black original surface demonstrated a yield stress, \( \sigma_y \), of 235.5 MPa; an ultimate tensile stress, \( \sigma_{UTS} \), equal to 362.9 MPa; and 13.7% reduction of area. More details of the static and fatigue tests results reported in [70] are summarised in Table 6-1 and in the form of an S-N diagram in Figure 6-1. The statistical analysis of the fatigue data to estimate the mean S-N relationship was performed according to the procedure described by ASTM [71]. A single-slope S-N curves were used because of the lack of adequate test results near the fatigue limit region to establish an accurate fatigue limit value. The scatter bands shown by the dash lines in Figure 6-1 refer to the 90% and 10% probabilities of survival, \( P_S \), determined under the hypothesis that \( \log N \) is normally distributed (Log-Normal distribution of number of cycles to failure for each stress
Fatigue life evaluation of simple details

level) assuming a 95% confidence level. The scatter ratio of stress amplitude, \( T_o \), (calculated for stress ranges at \( 2 \times 10^6 \) cycles to failure which intercept with the 10% and 90% scatter bands), for plain and notched specimens equals to 1.356 and 1.376, respectively. Despite the larger scatter observed in the results generated by testing samples containing a hole compared with that of the plain specimens, the statistical dispersion of both fatigue curves is comparable.

Figure 6-1 CA fatigue tests on River Cree wrought-iron specimens. Notched S-N curve refers to the gross cross-section area. (a) Shows the detail of notched specimen containing a central hole (mm) [70].

Table 6-1 Summary of experimental test results generated under static and CA fatigue loading [70]

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>t (mm)</th>
<th>R</th>
<th>m</th>
<th>UTS (MPa)</th>
<th>( \Delta \sigma^a ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>0.0</td>
<td>10.6</td>
<td>362.9</td>
<td>217.4</td>
</tr>
<tr>
<td>Notched</td>
<td>13</td>
<td>12.7</td>
<td>12.7</td>
<td>0.0</td>
<td>10.1</td>
<td>362.9</td>
<td>156.04</td>
</tr>
</tbody>
</table>

\(^a\) Fatigue strength at \( 2 \times 10^6 \) cycles to failure related to the net section area.
6.2.2 FE analysis

The commercial FE-package Abaqus/CAE v6.10 was used to model the specimen with a central hole considering the modelling recommendations given earlier in this chapter. The part was modelled using full-integration brick elements with five circular partitions around the hole perimeter with each partition creating 1 mm spacing to the adjacent one as illustrated in Figure 6-2. Through-thickness partitioning was also adopted to create sufficient stress readout points within the critical volume at the hole edge. One transverse end face of the model was restrained from movement in all directions and the other corresponding end face was subjected to tensile axial loading at various stress levels to simulate the experimental fatigue tests performed in [70]. To avoid rigid body motion, the loaded face was restrained against movement in its plane in vertical direction. The loading direction is denoted as ‘\(\sigma_{\text{applied}}\)’ in Figure 6-1 (a).

![Figure 6-2](image)

*Figure 6-2 Detail of the FE mesh used in the modelling of the wrought-iron specimens*

A fatigue-critical region along the perimeter of a rivet hole is shown in a close-up view sketched in Figure 6-3. The black dotted lines refer to the boundaries of a hemisphere centred at the rivet hole edge (marked at the notch tip in Figure 6-3) and having a radius equal to 1.54\(L\), representing the critical volume used when the volume method (VM) is adopted to
determine $\sigma_{\text{ave}}$ (average of linear-elastic principal stresses in the vicinity of the critical notch tip) [33, 4].

In such cases, to estimate $\sigma_{\text{ave}}$, the magnitude of linear-elastic maximum principal stresses or the nodes that lie within the critical volume (shaded in red in Figure 6-3) is obtained and averaged.

Similarly, when the area method formalisation of the TCD is used (AM), $\sigma_{\text{ave}}$ is determined by averaging the linear-elastic maximum principal stress values at the nodes which lie within a semi-circular area centred at the rivet hole edge (marked at the notch tip in Figure 6-3) and having a radius equal to 1.32$L$ [36].

6.2.3 Results and discussions

The fatigue life of the specimen shown in Figure 6-1 (a) was estimated based on the $L$ vs. $N_f$ relationship defined in Chapter 3 for old structural metals (see Figure 3-4). With regard to the S-N based fatigue life estimation, all four S-N classifications presented in Figure 3-1 (i.e., ‘Class D’, ‘Detail Category 71’, ‘WI-rivet Class’ and ‘modified Class B’) were used in conjunction with the nominal stresses obtained from the FE analysis at the net cross-section. The
obtained results are presented in the form of experimental number of cycles to failure, $N_f$, vs. estimated number of cycles to failure, $N_{f,e}$, as shown in the diagram in Figure 6-3. The dash lines shown in this figure refer to the experimental data constant amplitude scatter bands plotted to quantify the accuracy of the predictions.

Figure 6-4 shows that the VM and the AM estimates are within the constant amplitude scatter bands, but slightly on the conservative side in the high-cycle region. By contrast, the LM appears to yield the least accurate predictions (compared with predictions of other TCD formalisations) with only the results of the high-cycle region falling within the scatter of plotted experimental data. The PM estimates follow a similar pattern to that of the LM but with a higher degree of conservatism in the high-cycle region.

The chart in Figure 6-4 also makes it evident that the predictions of the S-N approach can be characterised with a high degree of conservatism with most of the medium- and high-cycle region results falling outside the CA scatter bands in the conservative side. Class WI-rivet and modified Class B resulted in the most conservative estimates in contrast to BS5400 Class D which provided the least conservative predictions. Overall, the TCD predictions (especially in the form of the AM and the VM) display much higher accuracy when compared with those of the S-N approach.

![Figure 6-4 Comparison of the TCD and S-N methods in fatigue life estimation of wrought-iron plates with a central circular hole under CA uniaxial loading](image)
Table 6-2 Error percentage range in predicted fatigue strength estimated using the TCD

<table>
<thead>
<tr>
<th>PM error %</th>
<th>LM error %</th>
<th>AM error %</th>
<th>VM error %</th>
</tr>
</thead>
<tbody>
<tr>
<td>-23 to 12</td>
<td>-22 to -1</td>
<td>-7.9 to 3.8</td>
<td>-2.2 to 8.6</td>
</tr>
</tbody>
</table>

* Negative value refers to nonconservative result in contrast to positive value which denotes conservative prediction.

Table 6-2 shows the error percentage in the estimated fatigue strength values based on the TCD compared with the corresponding experimental mean values. The error percentage is calculated as follows:

\[
\text{Error} \% = \left(\frac{\Delta \sigma_{\text{ave}} - \Delta \sigma_{\text{Exp}}}{\Delta \sigma_{\text{Exp}}}\right) \times 100
\]

where \(\Delta \sigma_{\text{ave}}\) (average reference stress value) is the predicted value of fatigue strength obtained using the TCD method, and \(\Delta \sigma_{\text{Exp}}\) refers to the corresponding fatigue strength value of plain specimens both determined considering the same number of cycles to failure. It can be observed that both the AM and the VM were capable of predictions with less than 10\% error, which is well within the typical error interval of ±20\% pertinent to the TCD as shown in the previous research [11, 30, 31, 69]. On the contrary, the PM and the LM predictions had good accuracy especially in the high-cycle regime and only exceeded the ±20\% error factor in one case which refers to the medium-cycle fatigue regime.

It is evident that even slight change in the value of the reference stress range can result in a much substantial deviation of \(N_f\) value. This is, as mentioned before, caused by the fact that the stress values in the S-N curve relationships (see Eqs. 3-1 and 3-2) are raised to the power of the inverse slope, \(m\), to determine the \(N_f\) value.

### 6.3 Flat wrought-iron specimens with one central circular hole

#### 6.3.1 Experimental data

Stier et al. [73] performed static and CA fatigue tests on wrought-iron specimens obtained from actual bridge structures in Germany. The investigated wrought-iron material from two different bridges built in 1880 and 1856 demonstrated a yield stress, \(\sigma_y\), ranging between 220.8 to 279 MPa, an ultimate tensile stress, \(\sigma_{\text{UTS}}\), ranging between 356.4 to 377 MPa, a range
Fatigue life evaluation of simple details

of 17.3 to 19.3% reduction of area and an average Young’s modulus of 132.8 GPa (ranging between a maximum value of 193.2 GPa and a minimum value of 84 GPa).

CA fatigue tests at a load ratio of, \( R \), equal to +0.1 were carried out on flat small-scale plain and notched (plates with one central hole) specimens and riveted double lap joint. As to the wrought-iron small-scale flat specimens, fatigue behaviour of two types of specimen were investigated; one containing a newly drilled central hole and the other performed on flat specimens with an original central hole. The outcome of the above investigation formed the basis for the German Federal Railway Code DS 804/6 [73]. More details of the above static and fatigue test results are provided in Table 6-3 [73].

In the following, fatigue analysis of the two types of notched specimens are presented separately. The first section covers fatigue analysis of samples with newly drilled central hole. Fatigue assessment of the specimens with original hole are investigated in the second part.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Hole diameter (mm)</th>
<th>t (mm)</th>
<th>( R )</th>
<th>( m )</th>
<th>UTS (MPa)</th>
<th>( \Delta \sigma ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>-</td>
<td>NA</td>
<td>0.1</td>
<td>6.02</td>
<td>356.4/377</td>
<td>187</td>
</tr>
<tr>
<td>Newly drilled hole</td>
<td>20</td>
<td>13.8-18.3</td>
<td>0.1</td>
<td>11.59</td>
<td>356.4/377</td>
<td>162</td>
</tr>
<tr>
<td>Original hole</td>
<td>20</td>
<td>13.8-18.3</td>
<td>0.1</td>
<td>3.33</td>
<td>356.4/377</td>
<td>78.5</td>
</tr>
</tbody>
</table>

\( a \) Fatigue strength at \( N_f = 2 \times 10^6 \) cycles to failure related to the net section area

6.3.2 Flat plate with newly drilled central circular hole

The cross-section of the flat specimens containing a newly drilled central circular hole had a thickness (t) of 13.8 and 18.3 mm, gross width ‘b’ of 57 and 70.4 mm and finally, total length of 500 mm. The experimental S-N data showing the fatigue behaviour of the above samples are presented in the form of an S-N diagram in Figure 6-5. To be consistent with case studies presented earlier, the above plain and notched fatigue curves are reproduced and presented as single-slope S-N curves. The details of the test specimens with a newly drilled central circular hole are also shown in the same figure.

The statistical reanalysis of the fatigue data to estimate the mean S-N relationship was performed according to the procedure described by ASTM in [71]. The above fatigue curves are estimated for a probability of survival, \( P_S \), equal to 50%. The scatter bands shown by
dashed lines in Figure 6-5 refer to the 90% and 10% probabilities of survival, determined under the hypothesis that Log N is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. The scatter ratio of the stress amplitude, \( T_o \), (calculated for stress ranges at \( 2 \times 10^6 \) cycles to failure which intercept with 10% and 90% scatter bands), for plain and notched specimens is 1.645 and 1.295, respectively. Slightly larger scatter is observed in the results generated by testing plain samples compared with that of notched specimens.

![Figure 6-5](image)

*Figure 6-5  Results of CA fatigue tests on wrought-iron flat specimens with a newly drilled central hole. Notched S-N curve refer to the gross cross-section area. Details of the tested specimens are shown in the above diagram with plate thickness ‘t’ having values of 13.8 and 18.3 mm [73]*

### 6.3.3 FE analysis

The commercial FE-package Abaqus/CAE v6.10 was used to model the flat plate with newly drilled central hole considering the modelling recommendations given in this chapter. One transverse end face of the model was restrained from movement in the X-, Y- and Z-directions and the other corresponding end face was subjected to tensile axial loading at various stress levels to simulate the experimental fatigue tests performed in [73]. To avoid rigid body motion,
Fatigue life evaluation of simple details

The loaded face was restrained against movement in its plane in the vertical direction. The loading direction is denoted ‘\( \sigma_{\text{applied}} \)’ in Figure 6-5. Details of the FE mesh used for the finite element analysis of the wrought-iron specimens with varying thickness and rivet hole diameter is shown in Figure 6-6.

![FE mesh used in the modelling of the wrought-iron specimens](image)

**Figure 6-6 FE mesh used in the modelling of the wrought-iron specimens**

### 6.3.4 Results and discussions

The fatigue life of the specimen shown in Figure 6-6 was estimated based on the TCD adopting the \( L \) vs. \( N_f \) relationship determined in Chapter 3 for old structural metals (see Figure 3-4). As to the S-N approach fatigue life estimation, ‘Class D’, ‘Detail Category 71’, ‘WI-rivet Class’ and ‘modified Class B’ mean S-N curves were used in conjunction with the nominal stress obtained from the FE model net section. Figure 6-7 summarises the results obtained from the fatigue life estimation performed using the S-N and the TCD methods for specimens with a newly drilled circular hole. The results are presented in the form of experimental number of cycles to failure, \( N_e \), vs. estimated number of cycles to failure, \( N_f \). The dash lines in Figure 6-7 refer to the experimental data constant amplitude scatter bands plotted to indicate the accuracy of the predictions when compared to the test results.
Fatigue life estimates obtained by the TCD method in the medium- and high-cycle fatigue regions show some degree of conservatism but mostly falling within the CA scatter bands. It is evident that the use of the PM, AM and VM resulted in very similar predictions. On the contrary, the LM predictions displayed the lowest level of conservatism in the high-cycle region with all the estimates being within the target error factor.

As to the accuracy of S-N method, the use of Class D and Detail Category 71 resulted in predictions more or less in line with the results obtained based on the Line method. To conclude this section, it is worth noting that, the Class WI-rivet as well as modified Class B, again, predicted rather similar fatigue lives with the highest degree of conservatism. Overall, the predictions for small and large cross-section specimens seemed to be predominantly identical.
6.3.5 Flat plates with original central circular hole

Test specimens used for this series of CA fatigue tests had the same cross-section dimension as those with newly drilled hole with plate thicknesses (t) equal to 13.8 and 18.3 mm, gross widths (b) equal to 57 and 70.4 mm and an overall length equal to 500 mm.

In Figure 6-8, the experimental data showing the fatigue behaviour of the samples with the original central circular hole are presented in the form of an $S$-$N$ curve. For the sake of consistency with previous case studies, the above plain and notched fatigue curves are reproduced and presented as single-slope $S$-$N$ curves.

The statistical analysis of the fatigue data to approximate the mean $S$-$N$ relationship was performed according to the procedure described by ASTM in [71]. The above fatigue curves are estimated for a probability of survival, $P_S$, equal to 50%. The scatter bands shown by dashed lines in Figure 6-8 refer to the 90% and 10% probabilities of survival, $P_S$, determined under the hypothesis that Log $N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level.

The scatter ratio of stress amplitude, $T_\sigma$, (calculated for stress ranges at $2 \times 10^6$ cycles to failure which intercept with 10% and 90% scatter bands) for plain and notched specimens is 1.645 and 2.631, respectively. Much higher dispersion of fatigue data is observed in the results generated by testing notched samples compared to plain specimens.

6.3.6 FE analysis

The specimen was modelled following the modelling recommendations given earlier in this chapter. As to the boundary condition, one transverse end face of the model was restrained from movement in the $X$-, $Y$- and $Z$-directions and the other corresponding end face was subjected to tensile axial loading at various stress levels to simulate the experimental fatigue tests performed in [73]. To avoid rigid body motion, the loaded face was restrained against movement in its plane in the vertical direction.
Figure 6-8 Results of CA fatigue tests on wrought-iron specimens with an original central hole. Notched S-N curve refers to the gross cross-section area. Details of the tested specimen are shown in the diagram (dimensions are in mm)

6.3.7 Results and Discussions

Fatigue life of the specimens with original central circular hole shown in Figure 6-8 were estimated based on the TCD method using the $L$ vs. $N_f$ relationship determined in Chapter 3 for old structural metals.

As to the S-N approach fatigue life estimation, all the mean S-N curves considered in this study, that is, ‘Class D’, ‘Detail Category 71’, ‘WI-rivet Class’ and ‘modified Class B’, were used in conjunction with the nominal stress obtained from the FE model for the net cross-section.

Figure 6-9 presents the experimental, $N_i$, vs. estimated, $N_{ie}$, fatigue life predictions obtained when the TCD and S-N methods are applied for fatigue life assessment of the wrought-iron specimens with a central circular hole. The dash lines shown in this figure refer to the constant amplitude scatter bands of the experimental data plotted to indicate the accuracy of the predictions. Fatigue life predictions obtained by applying the PM, AM and VM, even though always within the CA scatter bands, but appeared to be on the nonconservative side. However,
the LM predictions were seen to be outside the scatter bands with estimated fatigue strength values greater than the target error factor of ±20%. In overall, apart from the LM, the TCD predictions were within the scatter of the experimental data. In the contrary, the S-N method predictions which generally contain some level of inaccuracy, were seen to be within the CA scatter bands with acceptable degree of precision especially in the high-cycle fatigue regime. Class D and Detail Category 71 estimates were seen to always fall on the nonconservative side of the accuracy chart in contrast to predictions of the WI-rivet and modified Class B falling mostly on the conservative side.

Figure 6-9  Comparison of the TCD and S-N method fatigue life estimations of flat wrought-iron plates with an original central circular hole under CA uniaxial fatigue loading
6.4 Flat wrought-iron specimens with two circular holes

6.4.1 Experimental data

Mang et al. [53] performed CA fatigue tests at various stress ratios on part of a whole bridge as well as on small- and large-scale wrought-iron specimens recovered from two different bridges (Blumberg and Stahringen bridges built in 1887 and 1895, respectively). Members of the Blumberg Bridge, after a test on part of the whole bridge, was cut into smaller pieces to perform CA fatigue tests on different components and connections. The remaining wrought-iron material of bracings was used to punch specimens with new circular holes with diameter equal to 20 mm. Further specimens with original holes were salvaged from the remaining parts of main and longitudinal girders and tested at a stress ratio, $R$, equal to +0.1. Full detail of these specimens is presented in Figures 6-10 (a-b).

The investigated wrought-iron material demonstrated a yield stress, $\sigma_y$, of 284 MPa, an ultimate tensile stress, $\sigma_{UTS}$, of 390 MPa and a 24% reduction of cross-section area.

In Table 6-4 additional details of the reported static and fatigue test results on small-scale specimens with original and newly drilled holes are provided.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Hole diameter (mm)</th>
<th>t (mm)</th>
<th>$R$</th>
<th>$m$</th>
<th>UTS (MPa)</th>
<th>$\Delta \sigma$ a (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain b</td>
<td>-</td>
<td>NA</td>
<td>0.1</td>
<td>6.84</td>
<td>NA</td>
<td>193.6</td>
</tr>
<tr>
<td>Plain c</td>
<td>-</td>
<td>NA</td>
<td>0.1</td>
<td>6.02</td>
<td>356.4-377</td>
<td>187</td>
</tr>
<tr>
<td>Original &amp; Newly drilled hole</td>
<td>20-31</td>
<td>10-20</td>
<td>0.1</td>
<td>4.64</td>
<td>390</td>
<td>124</td>
</tr>
</tbody>
</table>

a Fatigue strength at $N_f = 2 \times 10^6$ cycles to failure related to net section area.

b Refers to wrought-iron plain S-N curve at $R = 0.1$ taken from [54].

c Refers to wrought-iron plain S-N curve at $R = 0.1$ taken from [73].

Mang et al. [53] also provides results of fatigue tests on plain wrought-iron specimens carried out by Brühwiler et al. [54] for further comparison of fatigue strength reduction effects due to punched holes as compared to the tested specimens with original holes.

The statistical analysis of the fatigue data to approximate the mean S-N relationship was performed according to the procedure described by ASTM in [71]. Single-slope S-N curves are used because of the lack of adequate test results near the fatigue limit region to establish an accurate fatigue limit value.
Fatigue life evaluation of simple details

Figure 6-10  Results of CA fatigue test on wrought iron specimens with two original and newly drilled holes. Notched S-N curve refers to the gross cross-section area. Detail of the tested specimen is shown in the diagrams (mm) [53]. (a) Shows plain S-N curve reported in [54]  (b) Refers to Plain S-N curve reported in [73]
Fatigue life evaluation of simple details

The scatter bands shown by dashed lines in Figures 6-10 (a-b) refer to the 90% and 10% probability of survival, $P_S$, determined under the hypothesis that Log $N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. The scatter ratio of the stress range, $T_\sigma$, (calculated for the stress ranges at $2 \times 10^6$ cycles to failure which intercept with 10% and 90% scatter bands) for plain S-N curves shown in Figures 6-10 (a) and 6-10 (b) equals to 1.379 and 1.645, respectively. Larger scatter is observed in the results generated by testing samples containing holes ($T_\sigma = 1.822$) compared to that of plain specimens which is typical of old structural metal.

6.4.2 FE analysis

The specimen was modelled following the modelling recommendations given earlier in this chapter. As to the boundary condition, one transverse end face of the model was restrained from movement in the X-, Y- and Z-directions and the other corresponding end face was subjected to tensile axial loading at various stress levels to simulate the experimental fatigue tests performed in [53]. To avoid rigid body motion, the loaded face was restrained against movement in its plane in the vertical direction. The loading direction is denoted ‘$\sigma_{\text{applied}}$’ in Figures 6-10 (a-b).

6.4.3 Results and Discussions

Fatigue life estimation based on the TCD adopting the $L$ vs. $N_f$ relationship determined for old structural metals was carried out on specimens with two different cross-sections chosen from details provided in Figures 6-10 (a) and (b). The first model had a gross width, $B$, hole diameter, $d$, and plate thickness, $t$, equal to 70, 20 and 10 mm, respectively, with a total length of 350 mm. The second model had $B = 80$ mm, $d = 31$ mm, $t = 20$ mm and finally, a total length of 350 mm. Since the wrought-iron material-specific plain S-N curve data for the Blumberg Bridge were not given in [53], structural wrought-iron plain S-N curves (at same stress ratio, $R$, equal to +0.1) from two different sources were adopted for the purpose of life estimation based on the TCD. The detail of the plain S-N curves is given in Table 6-4 and plotted in Figures 6-10 (a) and (b). This enables the opportunity to investigate the implications of a scenario where a general $L$ vs. $N_f$ relationship is used along with a material-non-specific plain S-N curve for fatigue life prediction through the TCD. Moreover, the fact that the above plain S-N curves
are estimated for structural wrought-iron from two different sources also provides the possibility to assess the sensitivity of the TCD predictions to the adopted plain S-N curve. As to the S-N approach fatigue life estimation, ‘Class D’, ‘Detail category 71’, ‘WI-rivet Class’ and ‘modified Class B’ S-N mean curves were used in conjunction with nominal stresses obtained using the results of the FE analysis at the net cross-section. Figure 6-11 summarise the predictions obtained for the two considered specimen configurations (small and large cross-sections) when the plain S-N curve shown in Figure 6-10 (a) was adopted for fatigue life assessment to the TCD method. On the other hand, Figure 6-12 shows the corresponding results when the plain S-N curve shown in Figure 6-10 (b) is used for fatigue life estimation based on the TCD. The results are presented in the form of experimental number of cycles to failure, \(N_i\), vs. estimated number of cycles to failure, \(N_f\). The dashed lines shown in Figure 6-11 and 6-12 refer to the experimental data CA scatter bands plotted to indicate the accuracy of the predictions. The fatigue life estimates presented in Figures 6-11 and 6-12 show a high level of accuracy in predictions of the TCD method irrespective of the plain S-N curve adopted for the calculation of the average reference stress value, \(\Delta\sigma_{ave}\). The above results confirm that all formalisations of the TCD were successful in predicting fatigue strength values well within the acceptable error interval of ±20%. The fatigue life estimates of the LM were seen to be the most accurate. In these figures, it can also be seen that the PM, LM and VM methods once more gave predictions which are very similar and slightly on the conservative side of the charts. The TCD method predictions were seen to be only slightly sensitive to the choice of the plain S-N curve. Moreover, the use of the \(L\) vs \(N_i\) relationship calibrated for wrought-iron material of Chepstow Bridge, in fatigue assessment of simple details seemed to have no adverse effects on the predictions. The TCD method appears to have predicted higher fatigue life for thicker specimens up to a factor of 1.6 in medium- and high-cycle regions. This phenomenon was not effectively captured in the predictions of the S-N method.

The above figures also make it evident that, once again, the WI-rivet and modified Class B predicted fatigue life with the highest degree of conservatism and mostly outside the scatter bands. In the medium- and high-cycle regime, only Class D and Detail Category 71 predictions were within the error target.
Fatigue life evaluation of simple details

Figure 6-11  Comparison of the TCD and S-N method in estimating fatigue life for flat wrought-iron plates with two original or drilled circular holes under CA uniaxial fatigue loading. The plain S-N curve in Figure 6-10 (a) was used for fatigue life estimation based on the TCD method.
Figure 6-12 Comparison of the TCD and S-N method in fatigue life estimation of flat wrought-iron plates with two circular holes under CA uniaxial fatigue loading. The plain S-N curve in Figure 6-10 (b) was used for fatigue life estimation based on the TCD method.
6.5 Wrought-iron specimens with six circular holes

6.5.1 Experimental data

Brühwiler et al. [74] published results of a series of CA fatigue tests on flat specimens obtained from the tension flange of parts recovered from a wrought-iron railway bridge built in 1882. These specimens had six longitudinally centrally aligned rivet holes. The rivets were carefully removed to avoid any damage to the original rivet holes. A total of fourteen specimens were tested on four different stress levels at a stress ratio, $R$, equal to +0.1. The tested specimens had a gross width, $B$, hole diameter, $d$, plate thickness, $t$, and distance between two adjacent holes equal to 90, 20, 13, and 160 mm, respectively. The total length of each specimen was equal to 930 mm. However, the static properties and plain material fatigue data for the wrought-iron material of the railway bridge were not given in [74].

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Hole diameter (mm)</th>
<th>$t$ (mm)</th>
<th>$R$</th>
<th>$\sigma_m$ (MPa)</th>
<th>UTS (MPa)</th>
<th>$\Delta\sigma$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain $^b$</td>
<td>-</td>
<td>NA</td>
<td>0.1</td>
<td>6.84</td>
<td>NA</td>
<td>193.6</td>
</tr>
<tr>
<td>Plain $^c$</td>
<td>-</td>
<td>NA</td>
<td>0.1</td>
<td>6.02</td>
<td>356.4-377</td>
<td>187</td>
</tr>
<tr>
<td>6 Original hole</td>
<td>20</td>
<td>13</td>
<td>0.1</td>
<td>3.33</td>
<td>NA</td>
<td>128.03</td>
</tr>
</tbody>
</table>

$^a$ Fatigue strength at $N_f = 2 \times 10^6$ cycles to failure related to the net section area. $^b$ Refers to wrought-iron plain S-N curve at $R = 0.1$ taken from [54]. $^c$ Refers to wrought-iron plain S-N curve at $R = 0.1$ taken from [73].

Hence, for the purpose of fatigue life analysis to the TCD method, two wrought-iron material plain S-N curves from different sources [20, 22] was selected (both at $R = +0.1$) which are estimated at the same stress ratio as the one used during testing of the above specimens. The detail of the fatigue S-N curve for flat plates with six circular holes as well as the plain S-N curves used in this case study are provided in Table 6-5 and plotted in Figures 6-13 (a) and (b). The statistical reanalysis of the fatigue data to approximate the mean S-N relationship was performed according to the procedure described by ASTM in [71]. A single-slope S-N curves are used because of the lack of adequate test results near the fatigue limit region to establish an accurate fatigue limit value and also to be in line with previous case studies presented in this thesis. The scatter bands shown by dashed lines in Figures 6-13 (a) and (b) refer to the 90% and 10% probability of survival, $P_S$, determined under the hypothesis that $\text{Log } N$ is normally distributed.
Fatigue life evaluation of simple details

(Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. The scatter ratio, $T_\sigma$, of the stress ranges at $2 \times 10^6$ cycles to failure for plain S-N curves shown in Figures 6-13 (a) and 6-13 (b) equals to 1.379 and 1.645, respectively. Larger scatter is observed in the results generated by testing samples containing holes ($T_\sigma = 1.935$) compared to that of plain specimens which is typical of old structural steel and wrought-iron material.

6.5.2 FE analysis

For the estimation of the linear-elastic stress field for fatigue life evaluation, the FE-package Abaqus/CAE v6.10 was used to create the FE model of the specimen containing six circular holes. The specimen was modelled using full-integration brick elements (C3D20) with five circular partitions around the hole perimeter with each partition creating 1 mm spacing to the adjacent one. Following the modelling recommendations given earlier in this chapter at least 4 layers of elements through the thickness and minimum of 24 elements in the perimeter of the holes were used to ensure smooth linear-elastic stress gradient in the vicinity of the rivet holes as well as creating adequate stress readout points.

For the boundary conditions, one transverse end face of the model was constrained from movement in the X-, Y- and Z-direction and the other corresponding end face was restrained against movement in its plane in vertical direction. This end face was also subjected to tensile axial loading at various stress levels to simulate the experimental fatigue tests performed in [74]. The loading direction is shown as $\sigma_{\text{applied}}$ in Figure 6-13 (a).
Fatigue life evaluation of simple details

Figure 6-13 Results of CA fatigue tests on wrought-iron specimens with six original holes. Notched S-N curve refers to the gross cross-section area. Details of the tested specimen are shown in the diagram. (a) Shows the plain S-N curve reported in [54] (b) Refers to the Plain S-N curve reported in [73]
6.5.3 Results and Discussions

Fatigue life estimation based on different formalisations of the TCD was carried out on the specimen shown in Figure 6-13 (a) using the $L$ vs. $N_i$ relationship determined for old structural metals. Since two different plain S-N curves are adopted for fatigue life predictions to the TCD as shown in Figures 6-13 (a) and (b), the associated outcome will be presented separately to show the difference in accuracy of estimates. This, once again, provides the opportunity to assess the capability of the TCD in fatigue life estimation of old structural metals when material-specific plain S-N curve of the investigated specimen is not available. In other word, the sensitivity of the TCD prediction to the choice of the plain S-N curve used for fatigue life assessment can be investigated.

For the case of the fatigue life estimation according to the S-N approach, ‘Class D’, ‘Detail Category 71’, ‘WI-rivet Class’ and ‘modified Class B’ mean S-N curves were used in conjunction with the nominal stresses obtained from the FE analysis results at the net cross-section to predict fatigue life of the specimens.

Figure 6-14 (a) represent the results of fatigue life predictions based on different formalisations of the TCD when the plain S-N curve given in Figure 6-13 (a) was employed to calculate the required $\Delta \sigma_{ave}$ value. Figure 6-14 (b), on the other hand, refers to the corresponding fatigue life estimates determined using the plain S-N curve plotted in Figure 6-13 (b). The results are presented in the form of experimental number of cycles to failure, $N_i$, vs. estimated number of cycles to failure, $N_{i,e}$. The dashed lines plotted in Figures 6-12 (a) and (b) represent the CA scatter bands of the experimental data and indicate the accuracy of the predictions.

Comparison of the fatigue life estimates in Figures 6-14 (a) and 6-14 (b) makes it evident that the TCD predictions were not significantly affected by the choice of the plain S-N curve. In other words, the fatigue life predictions were not highly sensitive to the use of a plain S-N curve for a structural wrought-iron from a different source obtained at the same stress ratio as the one used in the experimental tests. This confirms that, in cases where the actual material-specific plain S-N curve data of the investigated detail is not available, taking advantage of a plain S-N curve from a different source generated at the same stress ratio can still produce acceptable results within the target error factor.
Figure 6-14 Comparison of the TCD and S-N method in predicting fatigue life of flat wrought-iron plates with six circular holes under CA uniaxial fatigue loading. (a) Refers to the results when the plain S-N curve in Figure 6-13 (a) was used for life estimation to the TCD. (b) Refers to the corresponding results when the plain S-N curve in Figure 6-13 (b) was employed.
The above figures also show that the TCD was successful in predicting fatigue life within the usual target error in the high-cycle regime. However, in the medium- to low-cycle regions, the predicted results were seen to fall slightly outside the scatter bands of the experimental data and on the conservative side. For the latter, the fatigue strength predictions were slightly above the error interval of +20%, hence, resulting in estimates falling outside the CA scatter bands. The LM predictions were seen to be the most accurate especially in the medium- and high-cycle fatigue regime with fatigue strength predictions well within an error interval of ±12%.

Because fatigue analysis of bridge structural components and connections is mostly concerned with fatigue loading in the high-cycle region, the obtained results confirm that the TCD can be successfully applied to predict fatigue life in such situations. Another outcome worth highlighting here is that in the results summarised in Figures 6-14 (a) and (b) for the cases investigated so far, the PM, AM and VM were found to give identical predictions. This is an interesting observation since determination of $\Delta \sigma_{ave}$ required for fatigue assessment based on the TCD, is much simpler when the PM or the AM is applied as compared to the VM. However, further investigation on more complex details and/or in the presence of more complex loading conditions, which is undertaken in subsequent chapters, will provide a better overview.

In terms of the fatigue life predictions based on the traditional S-N method, as seen before, Class WI-rivet and modified Class B predictions were mostly outside the CA scatter bands on the conservative side in low- and medium fatigue regions. Predictions obtained by Detail Category 71 were seen to be very close to that of the VM but slightly more conservative. Both Class D and Detail Category 71 predictions in the medium- and high-cycle fatigue regime appeared to be within the CA scatter bands of the experimental data but in some cases on the nonconservative side.
6.6 Mild steel single lap joint

6.6.1 Experimental data

A series of experimental tests were carried out on mild steel material (3 meter long original double angle bracing member) removed from Trezoi Bridge in Portugal which is a steel riveted railway bridge [75]. Experimental tests included tension tests on plain specimens as well as CA fatigue tests on a number of single lap joints (SLJ) machined from the bracing material. The illustration presented in Figure 6-15 shows the geometry and dimensions of the investigated SLJ connection containing an original rivet with 12 mm shank radius.

![Configuration and dimensions of the investigated single lap joint (mm) [75]](image)

CA fatigue tests were carried out on a total of eight specimens at stress ratio, $R$, equal to +0.1 at various stress levels. In the majority of the cases, the specimens failed by initiation of a fatigue crack at the rivet hole and propagation of it on the Z-Y plane intersecting the rivet axis and parallel to loading direction (see Figure 6-15). The Trezoi Bridge tensile tests revealed a mild steel material with yield stress, $\sigma_y$, equal to 401 MPa, ultimate tensile stress, $\sigma_{UTS}$, equal to 464 MPa, Young’s modulus, $E$, equal to 210 GPa, Poisson’s ratio, $\nu$, equal to 0.27, a 23% elongation, and finally, 66% reduction of cross-section area at fracture. A summary of the tensile and CA fatigue tests results is provided in Table 6-6. Fatigue mean S-N curve of the above single lap joint is also plotted in Figure 6-16 together with the 10% and 90% CA scatter bands representing the dispersion of the test data.
Table 6-6 Summary of the experimental test results generated under static and CA fatigue loading

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Hole diameter (mm)</th>
<th>t (mm)</th>
<th>R</th>
<th>m</th>
<th>UTS (MPa)</th>
<th>$\Delta \sigma$ a (MPa)</th>
<th>$T_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain$^b$</td>
<td>-</td>
<td>NA</td>
<td>0.1</td>
<td>4</td>
<td>NA</td>
<td>185</td>
<td>1.467</td>
</tr>
<tr>
<td>Single lap joint</td>
<td>24</td>
<td>10</td>
<td>0.1</td>
<td>5</td>
<td>25</td>
<td>464</td>
<td>100.7</td>
</tr>
</tbody>
</table>

$^a$Fatigue strength at $N_f = 2 \times 10^6$ cycles to failure related to the net section area. $^b$Refers to BS 5400-10 Class B mean S-N curve taken from [28].

Figure 6-16 Results of CA fatigue tests on Trezoi Bridge mild steel single lap joint (SLJ) containing one original rivet. SLJ S-N curve refers to the gross cross-section area of A-B face (see Figure 6-15) [75]. Plain S-N curve refers to Class B mean curve [28].

The statistical analysis of the single lap joint fatigue data to estimate the mean S-N relationship was performed according to the procedure described by ASTM in [71]. A single-slope S-N curve was considered because of the lack of adequate test results near the fatigue limit region to establish an accurate fatigue limit value. According to detail classification rules provided in [28], the Class B mean S-N curve can be used to represent the fatigue behaviour of the plain material related to specimens of rolled steel structural plates and sections.
Since the Trezoi Bridge mild steel material-specific plain S-N curve was not known, the Class-B mean S-N curve (design plus two standard deviation) was adopted for fatigue life estimation based on the TCD method. The detail of the Class B mean S-N curve as well as CA scatter bands corresponding to 2.3% probability of failure are provided in Table 6-6 and plotted in Figure 6-16. To be consistent with the procedure adopted in previous cases, the CA fatigue limit at $10^7$ cycles to failure, suggested in the fatigue code, for Class B was not considered and the mean S-N curve of Class B and that of the single lap joint were mathematically extended further in the high-cycle regime for fatigue life estimation based on the TCD and S-N method.

The CA scatter bands for the single lap joint shown by dashed lines in Figure 6-16 refer to the 90% and 10% probability of survival, $P_S$, determined under the hypothesis that $\log N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. The scatter ratio of the stress ranges, $T_o$, (calculated for stress ranges at $2 \times 10^6$ cycles to failure), for plain and SLJ S-N curves shown in Figure 6-16 equals to 1.467 and 1.695, respectively.

### 6.6.2 FE analysis

To perform fatigue life analysis using the TCD method, the FE-package Abaqus/CAE v6.10 was used to create the 3-D finite element model of the single lap joint which consisted of two partially overlapping steel plates fastened by a standard rivet with round exposed heads on either end. The rivet shank was assumed to fully fit the hole space with no clearance which is typical product of hot-driven riveting process. A linear elastic steel material property was assumed for the analysis with a Young’s modulus and Poisson’s ratio values of 210 GPa and 0.27, respectively, which correspond to material properties obtained from experimental tensile tests.

20-noded brick elements (C3D20) with full integration were used to mesh both the upper and lower plate and the rivet. Following the modelling recommendations given earlier in this chapter at least 4 elements through the thickness and minimum of 24 elements in the perimeter of the holes were used. This is to create adequately refined mesh in the vicinity of the stress raiser (rivet hole) to ensure smoother linear-elastic stress transition as well as creating adequate stress readout points. Details of the FE mesh defined for the analysis of the single lap joint are shown in Figure 6-17.
Figure 6-17 FE mesh adopted in the finite element analysis of the SLJ

Several contact surfaces were modelled between contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave surface algorithm. A total of five contact pair surfaces were defined (one plate-to-plate, two rivet head-to-plate and two rivet shank-to-hole surface) considering Penalty friction formulation for tangential behaviour and default Hard Contact for normal behaviour. A coefficient of friction, $\mu$, equal to 0.3 was used in all simulations. The clamping force in the rivet of the tested SLJs was not reported in [75]. To assess capability of the TCD method in estimating the potential clamping force which may have existed in the rivets of the tested SLJ specimens, three different low rivet clamping force values of 2, 22 and 40 MPa were considered in the FE analyses. The above clamping forces were defined in the FE model at mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus which applies a concentrated force over a user-defined cross-section surface. For the boundary condition, the end face of the lower plate, A-B, was constrained in all directions. The corresponding end face of the upper plate, C-D, was restrained against translation along the Y- and Z-direction (see Figure 6-15). This boundary condition was selected to represent the experimental test condition performed in [75]. The multistep FE analyses consisted of a first step for application of only the rivet clamping force followed by the next steps where various remote tensile stress amplitudes were applied in conjunction with propagation of clamping stress effects.
6.6.3 Results and discussions

Fatigue life estimation based on all four different formalisations of the TCD was carried out on results of the FE analyses of the SLJ specimen adopting the $L$ vs. $N_I$ relationship derived in Chapter 3 for old structural metals. The TCD fatigue life estimation was performed for each case of rivet clamping force separately using the BS5400 Class B mean S-N curve as the plain material S-N curve.

As to the S-N approach fatigue life estimation, all the pertaining mean S-N curves plotted in Figure 3-1, that is, ‘Class D’, ‘Detail Category 71’ and ‘modified Class B’, were used in conjunction with nominal stresses obtained from the FE analysis for the specimen net cross-section.

The diagrams reported in Figures 6-18, 6-19 and 6-20 show the TCD accuracy when employed together with Class B mean S-N curve to predict fatigue life of single lap joint specimens with rivet clamping force of 2, 22 and 40 MPa, respectively, subjected to uniaxial tensile force. Fatigue life predictions based on traditional S-N methods are also presented in the above figures for comparison. The results are expressed as experimental number of cycles to failure, $N_f$, vs. estimated number of cycles to failure, $N_{f e}$. The CA scatter bands of the experimental data are also plotted using dashed lines indicating the accuracy of the lifetime estimates.

Relatively high degree of conservatism in the medium- and high-cycle fatigue regime can be observed in the TCD predictions shown in Figure 6-18. The PM, LM and AM predictions were seen to fall mostly outside the CA scatter bands in the medium- and high-cycle regime with error factor in the predicted fatigue strength ranging between +22% to +76%. The TCD predictions shown in Figure 6-18 indicates that a rivet clamping force larger than 2 MPa must have existed in the SLJ specimens tested in [75].

The TCD accuracy in predicting the number of cycles to failure for specimens with a 22 MPa rivet clamping force improved noticeably as shown in Figure 6-19. The estimates obtained by all the TCD formalisations were seen to be within the CA scatter bands. The VM predictions could be characterised with the highest degree of accuracy compared to other formalisations of the TCD with all fatigue strength predictions falling within an error factor of ±15%.
Figure 6-18 Comparison of the TCD and S-N method in fatigue life estimation of mild steel single-rivet single lap joint under CA uniaxial fatigue loading. Results refer to clamping force of 2 MPa

Figure 6-19 Comparison of the TCD and S-N method in fatigue life estimation of mild steel single-rivet single lap joint under CA uniaxial fatigue loading. Results refer to clamping force of 22 MPa
On the contrary, predictions made by the PM and AM, despite being inside the experimental data CA scatter bands, have not resulted in fatigue strength estimate within an error interval of ±20% especially in the high-cycle region.

However, the results plotted in Figure 6-20 for specimens with a 40 MPa rivet clamping force, show that, the VM, AM, LM and PM were capable of successfully predicting fatigue strength values in the medium- and high-cycle regions falling within an error interval of ±3%, ±10%, ±14% and ±18%, respectively. The results in Figure 6-20 indicate that, the fatigue life predictions by the VM can be characterised with the highest degree of accuracy.

The results obtained through the TCD method demonstrates that the single lap joints experimentally investigated in [75] had rivet clamping forces around 40 MPa. This, once again, confirms the predictive capabilities of the TCD method in capturing the fatigue behaviour of more complex details such as a SLJ with unknown rivet clamping force. Eccentric loading of the plates in SLJ specimens subjected to axial tension causes tilting of the rivet, uneven bearing of the rivet shank over the thickness of the hole and bending of the plates around the rivet head. This behaviour introduces secondary stresses which make this type of connections
more prone to fatigue. The TCD method is shown to be successful in capturing such effects by the results presented in Figures 6-18, 6-19 and 6-20. By contrast, the S-N approach gave similar predictions irrespective of the magnitude of the rivet clamping force in the rivet. This could be because the S-N method uses experimentally estimated S-N curves which are produced based on the experimental test results of details with variable rivet clamping force and corrosion levels. Hence, it was expected to see less accurate estimates by the S-N method since the tested SLJs were machined from bracing members of the bridge which are normally among the least stressed components. However, the results of Class D and Detail Category 71 appears to be within the CA scatter bands and is some cases close to the estimates obtained using the TCD methods. In overall, from the direct comparison between the fatigue life estimates, it can be concluded that the use of the \( L \) vs \( N \) relationship calibrated for structural wrought-iron did not result in inaccurate predictions by the TCD method.

6.7 Wrought-iron butt joint

6.7.1 Experimental data

De Jesus et al. [76] published the results of a series of tests performed on the wrought-iron material obtained from the Fão Bridge in Portugal. The roadway bridge was originally opened to traffic in 1892. During the last rehabilitation process of the bridge in 2007, seven of the side diagonals were replaced with new ones and the obtained material was used for experimental investigations. A series of tensile tests were carried out on 22 specimens. The tests revealed a yield stress, \( \sigma_y \), equal to 219.9 MPa; ultimate tensile stress, \( \sigma_{UTS} \), equal to 359.3 MPa; Young’s modulus, \( E \), equal to 198.7 GPa; Poisson’s ratio, \( \nu \), equal to 0.26, a 23.1% elongation, and a 13.2% reduction of the cross-section area at fracture. The experimental program also included CA fatigue tests on riveted and bolted butt joints prepared using the wrought-iron material obtained from the Fão Bridge. The connections were fabricated by drilling 24 mm diameter holes in the flat plates machined from the salvaged side angles and joined together using the same rivets used in the last rehabilitation of the bridge in 2007.
The plates used to fabricate the butt joint connections had a thickness of 5 or 8 mm. The rivets used in the specimen preparation had 22 mm original diameters, and the 2-mm clearance space between the rivet and the rivet hole was filled as a result of rivet expansion during the riveting process. The geometry of the above riveted joints is illustrated in Figure 6-21.

A total of 15 riveted butt joints were prepared and tested under CA uniaxial fatigue loading with a stress ratio, $R$, equal to zero. In Table 6-7, the obtained tensile test results are summarised. The plain material S-N curve presented in this table refers to a structural wrought-iron plain material mean S-N curve obtained at $R = 0.0$ found obtained from [70] and used in this case study for fatigue life estimation according to the TCD method.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Hole diameter (mm)</th>
<th>$t$ (mm)</th>
<th>$R$</th>
<th>$m$</th>
<th>UTS (MPa)</th>
<th>$\Delta \sigma$ (MPa)</th>
<th>$T_\sigma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain $^b$</td>
<td>-</td>
<td>NA</td>
<td>0.0</td>
<td>10.6</td>
<td>NA</td>
<td>217.4</td>
<td>1.356</td>
</tr>
<tr>
<td>Butt-join</td>
<td>24</td>
<td>5-8</td>
<td>0.0</td>
<td>6.82</td>
<td>359.3</td>
<td>162.7</td>
<td>1.534</td>
</tr>
</tbody>
</table>

$^b$ Fatigue strength at $N_f = 2 \times 10^6$ cycles to failure related to the net section area. $^b$ Refers to the mean plain S-N curve taken from [70]

The fatigue S-N data presented in [76] also include the results of CA fatigue tests on bolted wrought-iron butt joints with the same configuration as the one shown in Figure 6-21, only instead of rivets, 22 mm bolts torqued to generate 0 and 52 MPa clamping forces were used. The S-N data for riveted butt joint fall in between the results for bolted joints with or without rivet clamping force, which implies clamping force magnitudes between 0 and 52 MPa were present in the tested riveted butt joints.

The S-N behaviour of the riveted butt joints described above is plotted in Figure 6-22 together with the 10% and 90% CA scatter bands representing the scatter in the test results.

The statistical analysis of the butt joints’ fatigue data to approximate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM in [71]. A single-slope S-N curve was considered because of the lack of adequate test data near the fatigue limit region to establish an accurate fatigue limit value.

The River Cree Bridge wrought-iron plain S-N curve [70], determined at the same stress ratio as the butt joints experienced ($R = 0.0$), was adopted for the purpose of lifetime estimation based on the TCD. The detail of the above plain mean S-N curve as well as CA scatter bands corresponding to 90% and 10% probability of failure are provided in Table 6-7 and plotted in
Fatigue life evaluation of simple details

Figure 6-22. To be consistent with the procedure used in previous case studies, the CA fatigue limit was not considered and the mean S-N curves of plain specimens and butt joints were extended into the medium-cycle region for fatigue life calculations based on both the TCD and the S-N methods.

![Butt joint specimen geometry and dimensions (mm)](image)

*Figure 6-21  Butt joint specimen geometry and dimensions (mm) [76]*

The CA scatter bands for riveted butt joints shown by the dash lines in Figure 6-22 refer to the 90% and 10% probabilities of survival, $P_s$, determined under the hypothesis that $\log N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. The dispersion ratios of stress range, $T_o$, (calculated for stress ranges at $2 \times 10^6$ cycles to failure) for the plain specimens and the riveted butt joint shown in Figure 6-22, were found to be equal to 1.356 and 1.534, respectively. The statistical
dispersion of fatigue results obtained for riveted butt joints is slightly larger than but comparable with that of plain specimens.

**Figure 6-22** Results of CA fatigue tests on the Fão Bridge wrought-iron riveted butt joints. The butt-joint S-N curve refers to the gross cross-section area of the A-B face (see Figure 6-21) [76]. The plain S-N curve refers to the River Cree Bridge wrought-iron mean S-N curve obtained from [70]

### 6.7.2 FE analysis

To carry out fatigue assessment according to the TCD method, Abaqus/CAE v6.10 was used to create a 3-D FE model of the butt joint shown in Figure 6-21. Zero clearance between the rivet shank and the hole surface was assumed as was also the case in [76]. The wrought-iron material was assumed to have linear elastic behaviour for the analysis with Young’s modulus and Poisson’s ratio of 198.7 GPa and 0.26, respectively.

The model was meshed using full-integration brick elements according to the modelling recommendations suggested earlier in this chapter. Adequately refined mesh was adopted at the perimeter of the rivet hole in the inner plate, which was expected to be the fatigue-critical location in these connections. The detail of the FE mesh adopted in the analysis of the butt-joint is given in Figure 6-23.

Several contact surfaces were modelled between the contacting bodies using the standard surface-to-surface contact method with finite sliding and master-slave surface algorithm. A
total of 14 contact pair surfaces were created (four inner plate-to-cover plate, four rivet head-to-cover plate, four rivet shank-to-cover plate hole surface and two rivet shank-to-inner plate hole surface) considering the penalty friction formulation for tangential behaviour and default hard contact for normal behaviour. A coefficient of friction, $\mu$, equal to 0.3 was used in all simulations.

As a result of varying thickness in the plates used to prepare the riveted butt joint specimens in [76], two different models were created in this case study to cover both bounds of the specimen geometry; the thickness of all the plates in the first model was 5 mm, whereas that in the second model was 8 mm. With regards to the clamping force in the rivets, three different relatively low clamping force values of 20, 30 and 40 MPa were considered. These rivet clamping values were chosen because the fatigue results for riveted butt joints in [76] were observed to fall in between the results for similar bolted butt joints with 0 and 52 MPa clamping forces. This is an indication to expect rivet clamping forces lower than 52 MPa in the experimentally tested riveted butt joints. In the FE model, the clamping force was defined at the midlength of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus, which applies a concentrated force over a user-defined cross-sectional surface.
Fatigue life evaluation of simple details

Regarding the boundary conditions of the FE model, the end faces of both inner plates, A-B and C-D, were constrained in the Y- and Z-directions and were subjected to uniaxial tensile force to simulate the CA fatigue tests. To avoid rigid body motion, the nodes falling on the E-E’ line on both cover plates were restrained from movement in all directions. The E-E’ line is located on the Y-Z plane at the centre line of the model which corresponds to the transverse symmetry line in the model (see Figure 6-21). This boundary condition was selected to represent the experimental test condition in [76].

The multistep FE analyses consisted of application of the clamping force in the first step followed by subsequent steps where various remote tensile stress amplitudes were applied in conjunction with the propagation of the clamping force effects.

6.7.3 Results and discussions

Fatigue life estimation based on different formalisations of the TCD method was carried out on the results obtained from the FE analyses of the butt joint with 20, 30 and 40 MPa rivet clamping forces. The $L$ vs. $N_f$ relationship determined in previous chapters for old structural metal was adopted to determine the critical length value required for lifetime estimation based on the TCD. The fatigue life estimated was carried out for each case of rivet clamping force separately using the River Cree plain S-N curve plotted in Figure 6-24.

With regards to the S-N approach fatigue life estimation, all the mean S-N curves plotted in Figure 3-1 (i.e., ‘Class D’, ‘Detail Category 71’, ‘WI-rivet Class’ and ‘modified Class B’) were used in conjunction with the nominal stresses obtained from the FE analysis results at the critical detail net cross-section. The cross-section of the inner plate rivet hole was identified as the most fatigue-critical detail.

The TCD accuracy in fatigue life estimation of wrought-iron butt joints of varied plate thickness with three different rivet clamping force is summarised in Figure 6-24. Results for butt joints with 8 mm thick plates and 20, 30 and 40 MPa rivet clamping force are shown in Figures 6-24 (a), (b) and (c), respectively. Figures 6-24 (d), (e) and (f), however, correspond to the results for specimens with 5 mm thick plates and 20, 30 and 40 MPa rivet clamping force, respectively. The predictions according to the TCD were obtained using the plain material mean S-N curve for the wrought-iron material of the River Cree [70], while the value of the critical length was determined using the $L$ vs. $N_f$ relationship, as described in Figure 3-4. The
results are presented in the form of experimental number of cycles to failure, $N_i$, vs. estimated number of cycles to failure, $N_{i,e}$ with the experimental data CA scatter bands plotted using dashed lines to show the accuracy of the predictions. The TCD estimates for butt joints with 8 mm thick plates and a 20 MPa clamping force are shown in Figure 6-24 (a), although within the scatter bands, they were found to be slightly conservative compared with the corresponding results for butt joints with 5 mm thick plates (see Figure 6-24 (d)). Similar trends were observed when comparing the TCD predictions in Figure 6-24 (b) with Figure 6-24 (e) as well as Figure 6-24 (c) with Figure 6-24 (f) for 30 and 40 MPa rivet clamping forces, respectively. This trend in the predictions of the TCD suggests that slightly higher rivet clamping forces were present in the rivet shanks of butt joint specimens with 8 mm thick plates. This finding conforms well with the results of the experimental study performed in [39, 73] on riveted joints with 2-, 3- and 5-inch grip lengths. The findings of [39, 73] concluded that rivets with 5-inch grip length developed considerably greater clamping forces compared with 3-inch grip rivets and that, similarly, greater clamping forces were present in rivets with 3-inch grip length compared with 2-inch grip rivets. According to the TCD predictions shown in Figure 6-24, butt joint specimens with 8 mm thick plates had rivet clamping forces between 30 MPa and 40 MPa. This clamping force value in specimens with 5 mm thick plates was estimated to be approximately 20 MPa. The numerical investigation conducted on the same wrought-iron butt joints made from the Fão Bridge salvaged material also presented in [76] concluded that rivet clamping forces of approximately 22 MPa were present in the above specimens. This conforms well with the TCD predictions in this study confirming satisfactory levels of predictive capabilities for the TCD method. It is worth highlighting here that predictions of the PM, AM and VM were found to be similar. On the contrary, the LM predictions can be characterised as slightly nonconservative.

Regarding the fatigue life predictions based on the traditional S-N method, all four different detail classifications considered in this study resulted in very conservative estimates with most of the predictions in the medium- and high-cycle regions falling outside the experimental data CA scatter bands. The Modified Class B predictions were found to be the most conservative while the Class D fatigue life estimates showed the lowest degree of conservatism.
Fatigue life evaluation of simple details

Figure 6-24 Comparison of the TCD and S-N method in fatigue life prediction of wrought-iron butt joint under CA uniaxial fatigue loading with $R = 0$. Diagrams of (a), (b) and (c) refer to butt joints with 8 mm thick plates, and results of (d), (e) and (f) correspond to results of specimens with 5 mm thick plates.
6.8 Mild steel double lap joint

6.8.1 Experimental data

Parola, Chesson, and Munse [42] published the results of a series of static and CA fatigue tests performed on riveted double lap joints (DLJ) in 1965. The main objective of the above experimental research was to investigate the effects of bearing pressure on the fatigue strength of riveted joints. Bearing ratio was defined as the calculated ratio of the bearing pressure on the rivet shank to the tensile stress in the net cross-section of the plate.

Four different bearing ratios of 1.37, 1.83, 2.36 and 2.74 were considered. CA uniaxial fatigue tests were carried out on 120 DLJ specimens under stress ratios, $R$, equal to -1 (41 tests), 0.0 (48 tests) and +0.5 (31 tests). The majority of the DLJ specimens were tested with ‘as-driven’ clamping force in the rivets while for the rest of the specimens a reduced clamping force was used to investigate its combine effects with bearing pressure on fatigue strength. The rivet clamping force for a portion of the DLJ specimens were reduced by machining away most of the rivet heads or by pressing the rivet heads to slightly detrude the shank. Rotation of the rivet in the rivet hole for reduced clamping specimens was prevented by welding a bar to the rivet heads of two adjacent rivets transverse to loading direction.

The DLJ specimens had a configuration similar to that shown in Figure 6-25 with one centre-plate, two identical outer-plate and four rivets with 22.23 mm nominal diameter arranged in a square pattern. To achieve the required bearing ratios, four different DLJ specimen types were fabricated with main variables being the plate thickness (thus rivet grip length), joint width and rivet spacing. The rivet holes were match-drilled and their diameter was kept constant at a nominal value of 23.183 mm. The plate thicknesses were chosen carefully so that the combined thickness of both outer-plates would be larger than the thickness of the centre-plate. Thus, the centre-plate in all specimen types was fatigue-critical and in most cases, failure initiated at the first row of the rivet holes in the centre-plate and progressed transversely in both directions towards the edge and to the adjacent rivet hole. All the specimens were prepared using American structural mild steel material in accordance with ASTM designation A7 standard [78]. Mechanical properties of the steel material were determined in monotonic tensile tests on 203.2 mm flat steel coupons and are summarised in Table 6-8.
### Table 6-8: Mechanical properties of steel material used in DLJ specimens [42]

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>t, d (mm)</th>
<th>Yield stress (MPa)</th>
<th>UTS (MPa)</th>
<th>Elongation%</th>
<th>Reduction in area %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate</td>
<td>19.05</td>
<td>264.1</td>
<td>448.2</td>
<td>30</td>
<td>55.4</td>
</tr>
<tr>
<td>Rivet</td>
<td>22.23</td>
<td>204.1</td>
<td>403.4</td>
<td>26.5</td>
<td>-</td>
</tr>
</tbody>
</table>

The investigation by Parola et al. [42] concluded that increasing rivet grip length as result of increase in plate thicknesses causes higher fatigue life. This verified the conclusion that larger grip length is associated with greater rivet clamping forces. Another outcome worth mentioning is that, lower fatigue strength was obtained as the bearing ratio in specimens increased. However, since increase of bearing pressure is interrelated with increase of transverse rivet spacing, decrease of plate thickness and decrease of rivet grip length, it was difficult in the experimental investigation to isolate solely the effect of changing the bearing pressure from the effects of change in other variables.

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**Figure 6-25 Double lap joint dimensions and configurations (mm) [42]**
6.8.2 FE analysis

Because of the large number of experimental tests performed by Parola et al. [42], only the results of tests carried out on the DLJ specimens with bearing ratio of 1.37 subjected to CA fatigue loading at \( R = 0.0 \) were selected for assessing the predictive capability of the TCD in fatigue life estimation of such connections. The DLJ presented in Figure 6-25 shows the configuration and dimensions of these specimens in detail. The average magnitude of the ‘as-fabricated’ (normal clamping force) and reduced clamping force in each rivet of the above specimens were estimated to be equal to 253 and 76 MPa, respectively [42]. These values were estimated assuming a friction coefficient of 0.4 and rivet diameter of 23.81 mm (assuming that the rivet hole was filled completely during the hot-driven riveting process). The reason behind selection of a higher friction coefficient compared to other research was a somewhat rougher surface observed on the plates used to fabricate the DLJ specimens.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Hole diameter (mm)</th>
<th>( R )</th>
<th>( m )</th>
<th>Centre-plate UTS (MPa)</th>
<th>( \Delta \sigma ) (MPa)</th>
<th>( T_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain(^b)</td>
<td>-</td>
<td>-</td>
<td>4</td>
<td>NA</td>
<td>184.97</td>
<td>1.467</td>
</tr>
<tr>
<td>DLJ- Normal Clamping</td>
<td>23.183</td>
<td>0.0</td>
<td>4.63</td>
<td>448.2</td>
<td>150.76</td>
<td>1.819</td>
</tr>
<tr>
<td>DLJ- Reduced Clamping</td>
<td>23.183</td>
<td>0.0</td>
<td>3.9</td>
<td>448.2</td>
<td>105.87</td>
<td>1.498</td>
</tr>
</tbody>
</table>

\(^a\) Fatigue strength at \( N_f = 2 \times 10^6 \) cycles to failure related to the net section area. \(^b\) Refers to the BS 5400-10 Class B mean S-N curve [28]

A summary of the S-N data obtained from CA fatigue tests on DLJ specimens with normal and reduced rivet clamping force is presented in Table 6-9. The detail of the plain S-N curve given in the above Table refers to the BS 5400-10 Class B mean S-N curve (design plus two standard deviation) which was used in this case study for fatigue life estimation based on the TCD method. The detail of Class B mean S-N curve as well as the CA scatter bands corresponding to a 2.3% probability of failure are plotted in Figures 6-26 (a) and (b). To be consistent with the procedure adopted in previous cases, a CA fatigue limit at \( 10^7 \) cycles for Class B was not considered and the mean S-N curve of Class B and that of double lap joints were extended into the high-cycle regime for fatigue life calculations based on the TCD and the S-N method. The fatigue mean S-N behaviour of the investigated DLJ with bearing ratio of 1.37 is also plotted in Figures 6-26 (a) and (b) for specimens with normal and reduced clamping force, respectively. The statistical analysis of the DLJ fatigue data to approximate the mean S-N
relationship was performed according to the procedure described by ASTM in [71]. A single-slope S-N curve was considered because of the lack of adequate test results near the fatigue limit region to establish an accurate fatigue limit value. The CA scatter bands for double lap joints shown by dashed lines in Figures 6-26 (a) and (b) refer to the 90% and 10% probabilities of survival, \( P_s \), determined under the hypothesis that \( \log N \) is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level.

The scatter ratio of stress range, \( T_o \), (calculated for stress ranges at \( 2 \times 10^6 \) cycles to failure) for DLJ S-N curves associated with normal and reduced rivet clamping forces equals to 1.819 and 1.498, respectively. Larger scatter is observed in the results generated by testing double lap joints with normal clamping force. This scatter may be because of different factors such as eccentric loading of centre-plate, variable initial clamping force in the rivets, differences in hole filling by the rivet shanks and possible stress concentration caused by fabrication damage. On the contrary, despite larger scatter in the S-N data for DLJ with normal clamping force, the slope of the mean S-N curves for both normal and reduced rivet clamping force specimens is comparable.

The Abaqus/CAE v6.10 was used to build up and analyse the 3-D model of the DLJ with specifications shown in Figure 6-25. The clearance between rivet shank and the hole surface was assumed as zero since hot-driven riveting process was adopted to fabricate the specimens.

A linear-elastic behaviour was assumed for the mild steel material used in the FE analysis with a Young’s modulus and Poisson’s ratio value of 210 GPa and 0.3, respectively.

Full-integration brick elements were used to mesh all plates and rivets according to the modelling recommendations given earlier in this chapter. Since the centre-plate is fatigue-critical, refined mesh was adopted at the perimeter of the rivet hole for the centre-plate.

Detail of the DLJ FE mesh used for the analyses are shown in Figure 6-27.

Several contact surfaces were modelled between the contacting bodies using standard surface-to-surface contact method with finite sliding and master-slave surface algorithm. A total of 20 contact pair surfaces were created (2 centre-plate-to-outer-plate, 8 rivet head-to-outer-plates, 4 rivet shank-to-centre-plate hole surface and 8 rivet shank-to-outer-plate hole surface) considering Penalty friction formulation for tangential behaviour and the default
hard contact for normal behaviour. A coefficient of friction, $\mu$, equals to 0.4 was used in all simulations similar to [42] to estimate rivet clamping force.

![Graph](image)

**Figure 6-26** Results of the CA fatigue test on mild steel double lap joints (DLJ) under stress ratio, $R = 0$ [42]. DLJ S-N curves refer to the gross cross-section area of A-B face (see Figure 6-25). Plain S-N curve refers to the BS 5400-10 Class B mean S-N curve [28]
With regard to the applied clamping force in the rivets, two models were created with normal (253 MPa) and reduced (76 MPa) rivet clamping force. These rivet clamping values were defined at the mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus, which applies a concentrated force over a user-defined cross-sectional surface.

Regarding the boundary condition, the end faces of both outer-plates, C-D (on the Y-Z plane), were constrained in all directions. Face A-B of centre-plate (on the Y-Z plane) was restrained in the Z-direction and subjected to tensile forces at various levels to simulate the CA fatigue tests (see Figure 6-25). This boundary condition was selected to represent the experimental test condition adopted in testing riveted DLJ specimens [42]. The multistep FE analyses consisted of the first step for application of the clamping force followed by the subsequent steps where various remote uniaxial tensile stress amplitudes were applied in conjunction with the propagation of the clamping force effects.
6.8.3 Results and discussions

Fatigue life estimation based on different formalisations of the TCD was carried out on results of the FE analyses of DLJ specimens with normal and reduced rivet clamping force. The $L$ vs. $N_r$ relationship shown in Figure 3-4 determined for old structural wrought-iron was adopted to determine the critical length values required for lifetime estimates based on the TCD. The fatigue life estimation was determined for specimens with normal and reduced rivet clamping force separately using Class B mean S-N curve given in Table 6-9.

As to the S-N method fatigue life estimation, the pertaining mean S-N curves in Figure 3-1 (i.e., Class D, Detail Category 71 and Modified Class B) were used in conjunction with nominal stress obtained from the results of the FE analyses of DLJ specimen at the net cross-section.

The diagrams shown in Figures 6-28 (a-b) indicate the accuracy of the TCD method in estimating fatigue life of mild steel DLJ specimens subjected to CA fatigue loading. The above diagrams are presented in the form of the experimental, $N_r$, vs. estimated, $N_{r,est}$, fatigue life. Figures 6-28 (a) and 6-28 (b) correspond to the results predicted for DLJ specimens with normal and reduced rivet clamping force, respectively. It is evident that the TCD was very successful in predicting fatigue lives always falling within the experimental data CA scatter bands both for normal and reduced clamping force specimens. In case of specimens with normal clamping force, the PM, AM and VM estimated fatigue strength with maximum error interval of ±6%, ±13% and ±9%, respectively. Fatigue strength predictions of the LM were seen to be slightly on the nonconservative side but still within the scatter bands. Moreover, despite the large scatter observed in the experimental results for DLJ with normal clamping force, the TCD predictions were seen to have very low deviation from the mean S-N curve.

With regards to DLJ specimens with a 76 MPa rivet clamping force, the TCD predictions in the form of the PM, AM and VM were very similar and could be characterised with slight degree of conservatism, but always inside the CA scatter bands. In the contrary, the LM predictions were seen to be very successful with maximum error interval of ±3%. The overall accuracy levels shown by the TCD method is encouraging, since, from statistical point of view, the predictions cannot be expected to be more accurate than the experimental data.
Figure 6-28 Comparison of the TCD and S-N method in the fatigue life prediction of mild steel double lap joints under CA uniaxial fatigue loading. Results of (a) and (b) refer to DLJ specimens with normal and reduced clamping forces, respectively.
In comparison, the fatigue life prediction capability of the traditional S-N method was seen to suffer with high degree of conservatism especially for DLJ specimens with normal clamping forces in the high-cycle region. For the case of DLJ specimens with reduced rivet clamping force, Class D and Detail Category 71 estimates were within the CA scatter bands while Modified Class B predictions always fell outside the acceptable limits. Moreover, it was expected to obtain S-N method predictions with higher degree of conservatism for specimens with higher clamping force. This could be explained by the fact that the design S-N curves for riveted details are established using the results of a large set of experimental data on riveted specimens with varying conditions such as, varying rivet clamping forces, different corrosion levels, dissimilar service life, rivet hole misalignment, rivet defects.
6.9 Concluding remarks

In this chapter, the TCD method in the form of the PM, LM, AM and VM was used to determine the fatigue life of simple details subjected to CA fatigue loading. Fatigue life prediction was carried out on old structural parts and simple joints (wrought-iron and mild steel), such as flat plates with circular hole(s), single lap joint, butt joint and double lap joint connections. The S-N method, as a more commonly adopted fatigue assessment method by most codes of practice, was also used for fatigue analysis of the same details to allow for assessing the predictive capabilities of both approaches.

The TCD method was found to be successful in predicting the fatigue life in majority of the cases, falling within the experimental data CA scatter bands and consistently more accurate than the S-N method estimates. This was true regardless of the fact that the critical length vs. number of cycles to failure relationship, $L$ vs. $N_f$, employed in fatigue assessment was calibrated using plain and notched (flat plates with a central circular hole) S-N data obtained from the wrought-iron material of the Chepstow Bridge [38]. This $L$ vs. $N_f$ relationship was used in all the presented case studies in this chapter to allow fatigue life estimation based on the TCD method. The obtained results confirmed that the $L$ vs. $N_f$ relationship shown in Figure 3-4 could be adopted for fatigue assessment of simple details made of old structural wrought-iron and/or mild steel when the TCD method is employed without causing adverse effects on the accuracy of the predictions. This implies, in cases where there is none or inadequate material-specific fatigue and static material property available to determine case-specific $L$ vs. $N_f$ relationship, applying the $L$ vs. $N_f$ curve shown in Figure 3-4 could still result in accurate fatigue strength predictions within a typical error factor of ±20% for the TCD method.

It can also be concluded that, as confirmed by the accuracy charts presented in this chapter for simple joints (such as single lap joints, double lap joints and butt joints), the novel TCD method was capable of successfully estimating the magnitude of the rivet clamping force in the investigated connections. By contrast, the S-N method fatigue life predictions were seen to be sensitive to the detail classification and in the majority of cases falling outside the experimental scatter bands on the conservative side.

The results achieved here offer confidence in the safe and reliable application of the TCD method for fatigue life assessment of the full-scale riveted built-up girder in the next chapter to assess its predictive capability for such members.
7 Fatigue life evaluation of riveted built-up girders

7.1 Introduction

In the previous chapter, the TCD method was applied to predict the fatigue life of small-scale specimens such as flat plates with circular hole(s), single and double lap joints and butt joints. In this chapter, as the next step in achieving the objectives of this thesis; the TCD method applied in the form of the PM, LM, AM and VM is used to estimate fatigue life of full-scale riveted built-up bridge girders subjected to constant amplitude uniaxial cyclic bending. This would assess the applicability and ability of the TCD methodology when employed for fatigue analysis of more complex riveted bridge details.

In order to quantify the accuracy of the TCD predictions; the commonly applicable S-N approach for fatigue design and assessment (suggested in many fatigue design and assessment codes of practice) is used to create the comparison basis. Fatigue assessment based on the S-N approach is performed using four different S-N classifications, that is, Class D [28], Detail Category 71 [27], WI-rivet Class [31] and Modified Class B [28], where appropriate in conjunction with the nominal stresses obtained from the FE analysis at the net cross-section in the fatigue-critical location. The predictions made using the TCD and the S-N methods are compared against the experimental data to both quantify the differences in the estimates, and to determine the accuracy levels.

A total of six experimental fatigue tests on full-scale riveted built-up girders are obtained from the technical literature (technical reports, journal articles and conference proceedings) and considered in this chapter. A brief description of these tests with the key findings is also presented. The experiments are based on both wrought-iron and mild steel built-up girders. Care was taken only to include experiments performed on girders either salvaged from old metallic bridges, or intended to be used in such structures. The experimental fatigue data obtained for each case study was reanalysed according to the procedures described by ASTM [71] to estimate the S-N behaviour and the CA scatter bands.

In order to reduce the computational costs of the complex FE analysis when applied to calculate the required stress fields in the fatigue-critical details; a shell-to-solid sub-modelling technique in the Abaqus/CAE v6.10 [39] was used to model and analyse the built-up girders. This cost-effective modelling technique is a two-step analysis procedure. It uses the results of
an initial global shell model to drive a more finely meshed solid sub-model. It usually is used to study in detail a critical section of a model (for example, the central section of a simply supported beam subjected to a central point load) which comprises a small portion of the whole model. The solid submodel is used in this study to provide an accurately detailed picture of the stress distribution in the fatigue-critical detail.

Fatigue assessment based on the TCD method is performed by postprocessing the results obtained from the FE analysis of the shell-to-solid submodel. The predictions are presented alongside the corresponding estimates obtained using the S-N approach. The obtained results are shown in logarithmic base accuracy charts with experimental and estimated number of cycles to failure on the Y- and X-axis, respectively, delimited by two straight lines corresponding to the experimental data CA scatter bands.

7.2 Fatigue assessment of full-scale bridge girders

Bruhwiler et al. [41], in 1990, published the results of a series of CA fatigue tests on full-scale riveted bridge members. One of the objectives of the experiment was to perform fatigue tests of up to 20 million stress cycles on actual full-scale, full-length specimens which were not cut from existing members (this would avoid possible distortion of the element due to cutting). The configuration of two of these test specimens investigated in this study is presented in Figures 7-1 (a-b) showing the cross-section details, support locations, load application points and rivet hole size and spacings. In more detail, Figure 7-1 (a) refers to wrought-iron riveted built-up girders obtained after demolition of a city bridge after 100 years of service life (built in 1884) and subjected to road and street traffic combination. A total of six girders were tested under fatigue loading; two of which had obvious corrosion signs and the other four were just slightly corroded. Results of a series of tensile tests on the wrought-iron material revealed a yield stress, $\sigma_y$, equal to 239 MPa, ultimate tensile stress, $\sigma_{UTS}$, equal to 344 MPa, Young’s modulus, $E$, equal to 170-190 GPa and a 15% fracture strain.
The second series of fatigue tests were carried out on mild steel rolled girders with a riveted cover plate intended to be used for temporary railway bridges (Figure 7-1 (b)). These girders were 50 years old at the time of testing, but unused and in excellent condition. The girders were tested with cover plates placed in tension flange. The mild steel material demonstrated a yield stress, $\sigma_Y$, of 226 MPa, an ultimate tensile stress, $\sigma_{UTS}$, of 388 MPa, Young’s modulus, $E$, of 210 GPa and a 40% fracture strain [41].

### 7.2.1 CA Fatigue tests on wrought-iron riveted built-up girders of a road bridge

A total of six girders were tested under four-point bending with stress ratio, $R$, equal to +0.1. The geometry and the test set-up for the above tests are illustrated in Figure 7-1 (a). To determine whether girders contained any pre-existing fatigue cracks due to 100 years of service life, low-stress levels were selected for the initial tests. The stress was then increased...
Fatigue life evaluation of riveted built-up girders

and cycle count returned to zero for that girder if no crack was detected after 6 to 20 million cycles, and a run-out was recorded for the previous stress level. Fatigue failure was recorded when an increase of 0.2 mm in girder deflection was observed at mid-span due to a crack. The obtained test results are summarised in Table 7-1. In most of the cases fatigue crack was developed at or near the rivet hole in the tension flange in the constant moment region (See Figure 7-1). The study concluded that the corrosion level away from the rivet hole was not severe enough to reduce the fatigue strength of the girder to lower levels than that caused by the effects of a rivet hole. The detail of the plain S-N curves presented in the above table refer to wrought-iron plain S-N curves at \( R = +0.1 \) taken from [50, 69] which was used here for fatigue life estimation according to the TCD method to assess the sensitivity of the predictions to the choice of plain material S-N curve.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>( R )</th>
<th>( m )</th>
<th>UTS (MPa)</th>
<th>( \Delta\sigma ) (MPa)</th>
<th>( T_\sigma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain(^b)</td>
<td>14</td>
<td>-</td>
<td>0.1</td>
<td>6.84</td>
<td>NA</td>
<td>193.6</td>
<td>1.379</td>
</tr>
<tr>
<td>Plain(^c)</td>
<td>35</td>
<td>-</td>
<td>0.1</td>
<td>6.02</td>
<td>356.4-377</td>
<td>187</td>
<td>1.645</td>
</tr>
<tr>
<td>Built-up girders</td>
<td>11</td>
<td>19</td>
<td>0.1</td>
<td>4</td>
<td>344</td>
<td>102.1</td>
<td>1.673</td>
</tr>
</tbody>
</table>

\(^a\) Fatigue strength at \( N_f = 2 \times 10^6 \) cycles to failure related to the net cross-section. \(^b\) Refers to a structural wrought-iron plain S-N curve at \( R = +0.1 \) taken from [54]. \(^c\) Refers to structural wrought-iron plain S-N curve at \( R = +0.1 \) taken from [73]

The S-N behaviour of the riveted wrought-iron built-up girder described above is shown in Figures 7-2 (a) and (b), together with the 10% and 90% CA scatter bands representing the scatter in the test data. The CA scatter bands for riveted built-up girder is plotted using dashed lines in Figures 7-2 (a) and (b) refer to the 90% and 10% probabilities of survival, \( P_S \), determined under the hypothesis that \( \log N \) is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. Dispersion ratio of stress range, \( T_\sigma \), (calculated for stress ranges at \( 2 \times 10^6 \) cycles to failure), for the plain specimen S-N curves shown in Figures 7-2 (a) and 7-2 (b) equals to 1.379 and 1.645, respectively, and for riveted built-up girder S-N curve equals to 1.673. It appears that the \( T_\sigma \) value for fatigue data obtained for riveted built-up girder is slightly larger than the corresponding value for the plain material S-N data, but they remain comparable.

Table 7-1 Summary of experimental results on built-up girders under static and CA fatigue loading

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**Figure 7-2** Results of the CA fatigue tests on wrought-iron riveted girders [41]. Built-up and HEB 1000 rolled girder S-N data refer to the gross cross-section area (see Figures 7-1 (a-b)). The plain S-N curve in (a) refers to structural wrought-iron plain S-N curve at $R = 0.1$ taken from [54]. Plain S-N curve in (b) refers to structural wrought-iron plain S-N curve at $R = 0.1$ taken from [73].
The statistical analysis of the riveted wrought-iron built-up girder fatigue data to approximate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM in [71]. The fatigue data expressed as ‘run-out’ were not used in the process of S-N curve estimation. A single-slope S-N curve was considered because of the lack of adequate test data near fatigue limit region to establish an accurate fatigue limit value.

In Figures 7-2 (a) and (b), the fatigue test results for mild steel HEB 1000 rolled girders with riveted cover plate are also plotted. It can be noted that the fatigue test results of HEB 1000 girders are in good agreement with that of the wrought-iron built-up girders.

### 7.2.2 FE analysis of riveted built-up girder

The shell-to-solid submodelling technique in Abaqus/CAE v6.10 [39] was used to model and analyse the built-up girder. This is a technique that uses an interpolation of the results of an initial global shell model (normally with relatively coarse mesh) onto the driven nodes on the appropriate parts of the boundary of a more refined solid submodel [39]. This modelling technique was used to reduce the computational costs of the analysis since only local results and behaviour of a specific portion of the global model was of interest.

For creating the initial shell global model, the cross-section of the girder shown in Figure 7-1 (a) was transformed into an equivalent I-section with an equal second moment of inertia and total depth without taking into account the area reduction due to rivet holes. The adopted I-section consisted of equal size 150-mm wide, 8-mm thick compression and tension flanges and a 584-mm deep, 16.012-mm thick web section. The wrought-iron material was assumed to have linear elastic behaviour during the analysis with Young’s modulus and Poisson’s ratio values of 190 GPa and 0.3, respectively. The global model of the built-up girder was meshed using 8-noded shell elements (S8R) as shown in Figure 7-3. The adopted FE mesh consisted of about 1300 S8R type elements. Regarding the boundary conditions, the tension flange of the global model was constrained against translation in the X-, Y- and Z-directions at one end and in the X- and Y- directions at the other end along the shell edge to form a simple support condition. In a multistep analysis, the model was subjected to four-point bending at the locations illustrated in Figure 7-1 (a). The loading was applied as static point loads to generate stress values between 45 to 168 MPa at the extreme fibre of the tension flange in the constant
Fatigue life evaluation of riveted built-up girders

moment region. The resultant output file was used in the next step of the analysis to drive the shell-to-solid sub-model.

Figure 7-3 FE mesh used to model the shell global model of wrought-iron built-up girder

In order to create the shell-to-solid sub-model, a length of 260 mm of the fatigue-critical central section of the global shell model was cut and replaced with a solid submodel of the built-up girder using the cross-section geometry given in Figure 7-1 (a). The submodel consisted of two tension and two compression flanges, a web plate and four rivets as shown in Figure 7-4. The solid submodel was meshed using 20-noded solid elements (C3D20). 44 elements were used at the perimeter of the rivet holes in tension flanges; which is expected to be the fatigue-critical locations. Following the modelling recommendations mentioned in the previous chapters, five elements were used through the thickness of each angle leg. Details of the FE mesh used in the submodel are presented in Figure 7-5.

The kinematic conditions in the solid submodel were interpolated from the driving nodes along the edges of the global model, which were created after removing the 260-mm central section. With respect to the size of the centre zone, a default value of 10% of the maximum shell thickness (in this case equal to 1.6 mm) was used. The centre zone is defined as a width of a certain size at the interface of the shell global model-solid submodel within which the driving nodes exist. These driving nodes are the link between the global and local model and used to transfer the results from the global model to the local submodel [39].

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Figure 7-4. Solid submodel central section of the Built-up girder overlaid on its shell model

Figure 7-5. Close-up of the FE mesh used to model the solid sub-model. (Approximately $10^5$ C3D20 brick elements)

With regard to the submodel build-up, the clearance between the rivet shank and the hole surface was assumed to be zero. Several contact surfaces were modelled between contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave
surface algorithm. A total of 24 contact pair surfaces were created (4 angle leg-to-web plate, 8 rivet head-to-angle leg, 8 rivet shank-to-angle leg hole surface and 4 rivet shank-to-web plate hole surface) considering Penalty friction formulation for tangential behaviour and the default Hard Contact for normal behaviour. A coefficient of friction, \( \mu = 0.3 \), was used in all simulations.

Since no rivet clamping force value was reported in [41], two low clamping force values of 5 and 50 MPa were investigated. These values were selected as typical low clamping force values in old riveted structural members to assess the predictive capability of the TCD method in predicting the fatigue life, as well as the pre-tension stress magnitude in the rivet shanks.

In the FE model the clamping force was defined at the mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus which applies a concentrated force over a user-defined cross-sectional surface.

In the shell-to-solid submodel analysis, the boundary condition and the FE mesh for the global shell model remained unchanged.

The multistep shell-to-solid submodel FE analyses was carried out with the clamping force application in the first step followed by consequent steps where results of the shell global model was interpolated onto the solid submodel.

### 7.2.3 Results and Discussions

Fatigue life estimation based on different formalisations of the TCD method was carried out on the results of the shell-to-solid submodel analysis with 5 and 50 MPa rivet clamping forces. The \( L \) vs. \( N_f \) relationship determined in the previous chapters for old structural metal was adopted to determine the critical length values required for life estimation based on the TCD method. Since the plain S-N curve for the wrought-iron material used in the built-up girders was not available, the fatigue life was estimated using two different wrought-iron plain S-N curves; detail of the plain S-N curves is shown in Table 7-1 and plotted in Figures 7-2 (a) and 7-2 (b). The plain S-N curves were obtained from the technical literature for the structural wrought-iron tested at the stress ratio, \( R \), equal to +0.1 [50, 69].

Moreover, fatigue life estimation based on the S-N approach was performed using all the S-N classes presented in Figure 3-1 (i.e., ‘Class D’ [28], ‘Detail category 71’ [27], ‘WI-rivet Class’
[31] and ‘modified Class B’ [28]) in conjunction with the nominal stresses obtained from the FE analysis of the submodel at the outmost fibre of the tension flange at the net cross-section.

Figure 7-6 Comparison of the TCD and the S-N method fatigue life estimates for wrought-iron built-up girders under CA fatigue loading. (a) and (b) refer to the fatigue life results for girders with 50 and 5 MPa clamping forces, respectively, when the lifetime was estimated using the plain S-N curve taken from [54]. (c) and (d) refer to the corresponding results when the plain S-N curve taken from [73] was used for fatigue life estimation.

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Figures 7-6 (a) and (b) show the TCD accuracy when employed together with the plain S-N curve obtained from [54] to predict the fatigue life of riveted built-up girders with 50 and 5 MPa rivet clamping forces, respectively. On the other hand, Figures 7-6 (c) and (d) represent the corresponding results when the plain S-N curve obtained from [73] was employed to predict the fatigue life. In order to create a comparison basis, the fatigue life predictions based on traditional S-N methods are also presented in the above accuracy diagrams. The results are expressed as an experimental number of cycles to failure, \(N_f\), vs. estimated number of cycles to failure, \(N_{f,e}\) with the CA scatter bands of the experimental data plotted in dashed lines indicating the conservative and nonconservative extreme boundaries.

Comparison of the results in Figure 7-6 shows that higher accuracy was observed in the TCD method fatigue life estimates for girders with 5 MPa rivet clamping force as compared to girders with 50 MPa rivet clamping force. This indicates average rivet clamping force values of about 5 MPa or less in the rivets of the wrought-iron built-up girders tested by Bruhwiler et al. [41]. Another observation worth mentioning is that the fatigue life estimated using the PM, AM and VM formalisations appear to be comparable. The predictions of the LM method were found to be most nonconservative compared to other TCD formalisations. Additionally, it could be said that, the TCD fatigue life predictions were seen to be slightly sensitive to the choice of the plain S-N curve used for fatigue assessment. However, in the majority of the cases, the predictions were within the CA scatter bands and the change in the overall trends were insignificant.

Considering the fatigue life predictions based on the traditional S-N method, all the predictions were seen to be always inside the CA scatter bands. Predictions through the Detail category 71, Modified Class B and Wi-rivet were mostly on the conservative side while Class D predictions were found to be mostly on the nonconservative side of the accuracy chart. Overall, considering the results for girders with 5 MPa rivet clamping force, it could be concluded that the TCD method in the form of the PM, AM, and VM and the S-N method (Class D and Detail category 71) were capable of making acceptable estimates close to the best-fit line in the medium- or high-cycle fatigue regime. Furthermore, the fact that the TCD method was capable of estimating the magnitude of average rivet clamping stress in the rivet shanks of the built-up girders used in the original experimental tests is very promising.
7.2.4 CA Fatigue tests on mild steel rolled girders with riveted cover plate

A total of four HEB1000 rolled girders were tested under four-point bending with a stress ratio, \( R \), equal to +0.1. The geometry and the test set-up for the above girders is shown in Figure 7-1 (b). Failure criterion was considered as an increase of 0.2 mm in girder deflection due to an observed crack. At this stage, the number of cycle to complete failure was not very important since crack growth was observed to be very rapid. Some of the tested girders were repaired/strengthened after a fatigue test result was obtained and subsequently tested at the same or higher stress range to obtain additional cracks (fatigue test results) in regions outside the repaired zone. The obtained test results are summarised in Table 7-2. In most of the cases fatigue crack was developed at or near a rivet hole in the rolled girder in the constant-moment central region. The detail of the plain S-N curve presented in the above table refers to BS 5400 Class B mean S-N curves [28] which corresponds to mild steel plain material and was used here for fatigue life estimation based on the TCD.

Table 7-2 Summary of experimental test results on mild steel rolled girders under static and CA fatigue loading [41]

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>( R )</th>
<th>( m )</th>
<th>UTS (MPa)</th>
<th>( \Delta \sigma ) (MPa)</th>
<th>( T_\sigma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain(^b)</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>4</td>
<td>-</td>
<td>184.97</td>
<td>1.467</td>
</tr>
<tr>
<td>Mild steel rolled girder</td>
<td>8</td>
<td>21</td>
<td>0.1</td>
<td>4</td>
<td>388</td>
<td>93.21</td>
<td>1.673</td>
</tr>
</tbody>
</table>

\(^a\) Fatigue strength at \( N_f = 2 \times 10^6 \) cycles to failure related to the net section area. \(^b\) Refers to BS5400-10 Class B mean S-N curve taken from [28]

The S-N behaviour of the HEB 1000 rolled girder with riveted cover plates described above is plotted in Figure 7-7 together with the 10% and 90% CA scatter bands representing the scatter in the test data. The scatter bands for HEB 1000 S-N data shown by dashed lines in Figure 7-7 refer to the 90% and 10% probabilities of survival, \( P_s \), determined under the hypothesis that \( \log N \) is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. Dispersion ratio of stress range, \( T_\sigma \), (calculated for stress ranges at \( 2 \times 10^6 \) cycles to failure), for plain specimen S-N curve equals to 1.467 and for the rolled girder’s S-N curve equals to 1.673. The statistical dispersion of fatigue results obtained for the rolled girder is slightly larger but comparable to that of the plan specimens.
Fatigue life evaluation of riveted built-up girders

Figure 7-7 Results of CA fatigue test on mild steel rolled girder with riveted cover plate [41]. Built-up and HEB 1000 girder S-N data refer to the gross cross-section area (see Figure 7-1 (a-b)). The plain S-N curve refers to BS 5400-10 Class B mean S-N curve [28]

The statistical analysis of fatigue data for the rolled girder with riveted cover plate to approximate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM in [71]. The fatigue data expressed as ‘run-out’ were not used in the process of S-N curve estimation but plotted in Figure 7-7. A single-slope S-N curve was considered because of the lack of adequate test data near the fatigue limit region to establish an accurate fatigue limit value.

One of the main observations in [41] was that, wrought-iron riveted girders, even after 100 years of service life, still exhibit similar fatigue strength comparable to that observed for mild steel elements that have never been in service. In Figure 7-7 the results of fatigue test on wrought-iron built-up girders are also plotted. It can be noted that HEB 1000 fatigue test results are in close agreement with that of wrought-iron built-up girders.
Fatigue life evaluation of riveted built-up girders

FE analysis of rolled girder with riveted cover plate

The shell-to-solid submodelling technique in Abaqus/CAE v6.10 was employed to model and analyse the rolled girder with a riveted cover plate. The initial shell global model was created by transforming the cross-section shown in Figure 7-1 (b) to an equivalent I-section with the same second moment of inertia, total depth and neutral axis location, without considering the area reduction due to rivet holes. The above I-section consisted of a 300-mm wide, 36-mm thick compression flange, a 967.29-mm wide, 18-mm thick tension flange and a 964-mm deep, 18.003-mm thick web plate. Linear elastic behaviour was assumed for the mild steel material during the analysis with Young’s modulus and Poisson’s ratio values of 210 GPa and 0.3, respectively. The global model was meshed using 8-noded shell elements (S8R) as shown in Figure 7-8. The global model FE mesh consisted of about 2600 S8R type elements. As to the boundary condition, the tension flange of the global model was constrained against translation in the X-, Y- and Z-directions at one end and in the X- and Y- directions at the other end along the shell edge to form a simple support.

![Figure 7-8](image)

Figure 7-8 The FE mesh used to model the shell global model of mild steel HEB1000 rolled girders

In a multistep analysis, the model was loaded in four-point bending at the locations illustrated in Figure 7-1 (b). The loading was applied as static point loads to generate stress values between 50 to 185 MPa in the extreme fibre of the tension flange in the constant moment region. The resultant output data file was used in the next step of the analysis to drive the shell-to-solid submodel.
As shown in Figure 7-9, the shell-to-solid submodel was created by removing 720 mm of the critical central section of the global shell model and replacing it with solid model of the rolled girder with riveted cover plate. The submodel consisted of a HEB 1000 rolled beam, a cover plate and 8 rivets (4 on each side) with the dimensions specified in Figure 7-1 (b). Fully-integration 20-noded solid elements (C3D20) were used to mesh the submodel. 40 elements were used at the perimeter of the rivet/rivet holes in tension flange and cover plate, which are the potential fatigue-critical locations. Following the FE modelling recommendations mentioned in the previous chapters, five and eight elements were used through the thickness of the HEB tension flange and the cover plate, respectively. Detail of the FE mesh used in the solid submodel is presented in a close-up image in Figure 7-10.

The nodes along the edges of the global model which were created after cutting 720 mm of the central section were used as driving nodes to interpolate the results of shell global model.
Fatigue life evaluation of riveted built-up girders

to the solid submodel. As to the size of the centre zone, a default value of 10% of the maximum shell thickness (in this case equal to 3.6 mm) was used. As to the submodel modelling, the clearance between the rivet shank and the hole surface was assumed equal to zero. Several contact surfaces were modelled between the contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave surface algorithm. A total of 33 contact pair surfaces were created (1 rolled girder-to-cover plate, 8 rivet head-to-rolled girder, 8 rivet head-to-cover plate, 8 rivet shank-to-rolled girder rivet hole surface and 8 rivet shank-to-cover plate rivet hole surface) considering Penalty friction formulation for tangential behaviour and default Hard Contact for normal behaviour. A coefficient of friction, $\mu$, equals to 0.3 was used in all simulations.

![Figure 7-10](image)

_Figure 7-10 Close-up of the FE mesh used to model the solid submodel. (136,000 C3D20 brick elements)_

Since no rivet clamping force was reported in [41], two relatively moderate to high rivet-clamping force values of 100 and 150 MPa were considered to assess the predictive capability of the TCD method. These values were assumed since rolled girders were never put in service and hence expected to have retained their original rivet clamping forces. In the FE model, the clamping force was defined at the mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus, which applies a concentrated force over a user-
defined cross-sectional surface. In the shell-to-solid submodel analysis, the boundary condition and FE mesh for the global shell model remained unchanged. The multistep shell-to-solid submodel FE analysis was carried out with application of just the clamping force in the first step followed by subsequent steps where results of the global shell model was interpolated onto the solid submodel.

Results and discussions

The TCD method in the form of the PM, LM, AM and VM was used to estimate fatigue life of rolled girder with riveted cover plate. The $L$ vs. $N_f$ relationship for old structural metal determined in previous chapters was adopted to calculate the critical length values required for lifetime estimation based on the TCD. The BS 5400-10 class B mean S-N curve, representative of mild steel plain material S-N data [28], plotted in Figure 7-7 was used for fatigue life prediction to the TCD method. Additionally, the S-N approach, as proposed in three different commonly used fatigue design and assessment codes of practice for riveted structures (i.e., ‘Class D’ [28], ‘Detail category 71’ [27], and ‘modified Class B’ [27]), were used in conjunction with the nominal stresses obtained from the FE analysis results at the outmost fibre of the cover plate at the net cross-section to predict fatigue life at various stress range levels.

Figures 7-11 (a) and (b) show the TCD method accuracy in estimating fatigue life of mild steel rolled girders subjected to four-point bending. The above results are plotted in the form of experimental number of cycles to failure, $N_f$, vs. estimated number of cycles to failure, $N_{f,e}$ with the CA scatter bands of experimental data plotted using dashed lines to indicate the accuracy of the predictions. The results in Figures 7-11 (a) and (b) correspond to specimens with 100 and 150 MPa rivet clamping forces, respectively. In order to create a comparison basis, the fatigue life predictions based on the traditional S-N method are also plotted in the above accuracy diagrams. The rivet hole edge in the cover plate and tension flange at the cover plate-rolled girder tension flange interface was considered to be the fatigue-critical location.
Figure 7-11 Comparison of the TCD and the S-N method fatigue life estimation of mild steel rolled girder with a riveted cover plate under CA fatigue loading. (a) and (b) refer to results of the specimen with 100 and 150 MPa rivet clamping force, respectively.
The results in Figures 7-11 (a) and (b) make it evident that different formalisations of the TCD predicted fatigue life always falling within the experimental data CA scatter bands in the medium- and high-cycle fatigue regime. In more detail, in case of specimens with 100 MPa rivet clamping force, the use of the PM, AM, LM, and VM predicted fatigue strength values between ±22%, ±20%, ±20% and ±16% error intervals, respectively. The corresponding error intervals for specimens with 150 MPa rivet clamping force was between ±16%, ±32%, ±14% and ±11%, respectively. It can be noted that the increase in the rivet clamping force to 150 MPa resulted in more accurate estimates by the TCD in the medium- and high-cycle fatigue, but still slightly on the conservative side of the accuracy diagram. Thus, by comparing the TCD predictions plotted in Figures 7-11 (a) and (b), it is sensible to conclude that, average rivet clamping forces slightly greater than 150 MPa existed in the mild steel rolled girders tested by Bruhwiler et. al. [41]. Another outcome worth noting here is that, the AM and the VM were noted to produce comparable fatigue lives with maximum difference of about 2 to 4% in the obtained results.

As to the fatigue life predictions based on the traditional S-N method, all the predictions appeared to be within the CA scatter band. The modified Class B estimates can be characterised by the highest degree of conservatism while Class D produced results mostly on the nonconservative side. Predictions of Detail category 71 were seen to be the most accurate of the S-N method, despite being marginally on the conservative side in the medium-cycle fatigue regime, but comparable with the TCD predictions for specimens with 150 MPa rivet clamping force.

In overall, the TCD method was successful in estimating fatigue life for riveted bridge girders with acceptable level of accuracy close to the best-fit line in the medium- and high-cycle fatigue regime. The ability of the TCD method in estimating the magnitude of the clamping stress in rivets of the rolled girder with riveted cover plate used in the fatigue tests by Bruhwiler [41] is encouraging.

The S-N method fatigue life predictions were sensitive to the choice of detail classification.
Fatigue life evaluation of riveted built-up girders

7.3 Fatigue assessment of the River Cree bridge built-up cross-girders

7.3.1 Experimental data

Burdon [70] carried out a series of static and constant amplitude fatigue tests on small size wrought-iron specimens (plain and notched) and full-scale cross-girders obtained from the River Cree Viaduct located in Scotland. The above tests were part of an additional effort to a maintenance procedure performed in 1957 which was intended to renew some of the timber piers of the viaduct after 100 years of service life. It was decided to also evaluate the working stresses in the wrought-iron superstructure of the viaduct as well as assessing whether the material was still capable of carrying such loads.

Static tensile tests were performed on wrought-iron specimens cut from cross-girders and cover plates. No significant difference was observed suggesting that the overall quality of the wrought-iron used along the viaduct in different members was fairly uniform. The mean values obtained in the tensile tests on specimens with black original- and fully-machined surface are quoted in Table 7-3.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Yield stress (MPa)</th>
<th>UTS (MPa)</th>
<th>Elongation%</th>
<th>Reduction in area %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black original surface</td>
<td>235.5</td>
<td>362.9</td>
<td>11.4</td>
<td>13.7</td>
</tr>
<tr>
<td>Fully machined surface</td>
<td>250.9</td>
<td>395.3</td>
<td>31.8</td>
<td>43.1</td>
</tr>
</tbody>
</table>

A total of three cross-girders were used in fatigue testing under four-point bending with a stress ratio, $R$, equal to zero. The geometry, loading configurations and test set-up for tests on the above cross-girders are illustrated in Figure 7-12. One of the cross-girders suffered from a web buckling in the initial test preparation stages. Hence, CA fatigue tests were carried out on only two of the remaining full-size cross-girders. Low stress levels were selected for the test on each cross-girder but it was then increased and cycle count returned to zero for that girder if no crack was detected after $10^6$ cycles and a run-out was recorded for the previous stress level. The obtained test results are summarised in Table 7-4. As expected, fatigue crack was developed around the rivet hole in the tension flanges in the constant-moment region in cases where fatigue failure occurred. The detail of the River Cree wrought-iron plain S-N curve obtained through CA fatigue tests at $R = 0$ on flat specimens are also
Fatigue life evaluation of riveted built-up girders

reported in Table 7-4 which was used in this case study for fatigue life estimation according to the TCD method.

Table 7-4 Summary of the experimental test results on River Cree girders under static and CA fatigue loading [70]

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>R</th>
<th>m</th>
<th>UTS (MPa)</th>
<th>Δσ <em>a</em> (MPa)</th>
<th>T_σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>12</td>
<td>-</td>
<td>0.0</td>
<td>10.6</td>
<td>362.9</td>
<td>217.4</td>
<td>1.356</td>
</tr>
<tr>
<td>Built-up girders</td>
<td>4</td>
<td>19</td>
<td>0.0</td>
<td>11.7</td>
<td>362.9</td>
<td>139.6</td>
<td>1.239</td>
</tr>
</tbody>
</table>

*Fatigue strength at N_f = 2 × 10^6 cycles to failure related to net section area (see Figure 7-12)*

The experiments concluded no significant reduction in the static and fatigue strength of River Cree wrought-iron despite 100 years of service life indicating negligible working stresses. Another outcome of the investigation was the proposed use of a ratio between 0.41 to 0.45 of the UTS to estimate the fatigue strength of the material in cases where sufficient material for fatigue testing is not available. Hence, multiplying the above ratio with the UTS determined using static tests would provide the fatigue strength at 10^6 cycles for larger components. Similar fatigue strength at 5 × 10^6 cycles life was suggested to be approximated using a reduced factor of between 0.31 and 0.39.

![Figure 7-12 Detail of River Cree wrought-iron riveted built-up girder (mm) [70]](image-url)
Fatigue life evaluation of riveted built-up girders

Figure 7-13  Results of the CA fatigue tests on River Cree wrought-iron built-up girders [70]. The built-up girder S-N data refer to the gross cross-section area (see Figure 7-12). The plain S-N curve refers to the River Cree wrought-iron material tested at $R = 0$ [70]

The experimental S-N data showing the fatigue behaviour of the River Cree cross-girders are presented in the form of S-N diagrams in Figure 7-13. The CA scatter bands shown by dashed lines in Figure 7-13 refer to the 90% and 10% probabilities of survival, $P_S$, determined under the hypothesis that $\log N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. Dispersion ratio of stress range, $T_o$, (calculated for stress ranges at $2 \times 10^6$ cycles to failure) for the plain specimen S-N curves shown in Figure 7-13 equals to 1.356 and for the River Cree cross-girder S-N curve to 1.239. The statistical dispersion of fatigue results obtained for cross-girder S-N data is slightly smaller, which is because of very limited number of reported fatigue data.

The statistical analysis of the riveted wrought-iron built-up girder fatigue data to estimate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM [71]. The fatigue data expressed as ‘run-out’ were used in the process of S-N curve estimation because of the lack of adequate fatigue data and also since the run-out
data corroborated closely with the other available fatigue data. A single-slope S-N curve was considered because of the lack of sufficient test data near fatigue limit region to establish an accurate fatigue limit value.

### 7.3.2 FE analysis of River Cree wrought-iron built-up cross-girders

The cost-effective shell-to-solid submodelling technique in Abaqus/CAE v6.10 was used to model and analyse the River Cree cross-girder. The initial shell global model was created by transforming the cross-section shown in Figure 7-12 to an equivalent I-section with the same second moment of inertia, total depth and neutral axis location, without considering the area reduction due to the rivet holes. The above I-section consisted of an equal size 159-mm wide, 14-mm thick compression and tension flanges and a 328-mm deep, 22.5-mm thick web plate. A linear elastic behaviour was assumed for the wrought-iron material during the analysis with a Young’s Modulus and Poisson’s ratio values of 190 GPa and 0.3, respectively. The global model was meshed using 8-noded shell elements (S8R) as shown in Figure 7-14. The global model FE mesh consisted of about 1,600 S8R type elements. As to the boundary condition, the tension flange of the global model was constrained against translation in the X-, Y- and Z-directions at one end and in the X- and Y-directions at the other end along the shell edge.

In a multistep analysis, the global model was subjected to four-point bending at the locations illustrated in Figure 7-12. The loading was applied as static point loads to generate stresses between 107 MPa and 150 MPa in the outmost fibre of the tension flange in the constant-moment region. The resultant output data file was used in the next step of the analysis to drive the shell-to-solid submodel.

The solid submodel superimposed on the shell global model is shown in Figure 7-15 which was created by removing a length of 305 mm of the global model central section and replacing it with solid submodel of the cross-girder. The submodel consisted of two tension and two compression flange angles, a web plate and six rivets with the dimensions specified in Figure 7-12. Brick elements (C3D20) were used to mesh the solid submodel. 44 elements were used at the perimeter of the rivets/rivet holes in tension flanges, which is expected to be the critical hot-spot. Following the modelling recommendations mentioned in the previous chapters, seven elements were used through the thickness of the tension flanges’ vertical legs. Detail of the FE mesh used in the solid submodel analysis is presented in a close-up shown in Figure 7-16.
Fatigue life evaluation of riveted built-up girders

Figure 7-14 FE mesh used to model the shell global model of River Cree cross-girder

Figure 7-15 Solid submodel and a close-up of the central section tension flange of the River Cree cross-girder overlaid on the shell global model
The kinematic conditions in the solid submodel were interpolated from the driving nodes along the edges of the global model which were created after removing the 305-mm central section. As to the size of the centre zone, a default value of 10% of the maximum shell thickness (in this case equal to 2.2 mm) was used.

As to the submodel modelling, the clearance between the rivet shank and the hole surface was assumed equal to zero. Contact surfaces were modelled between the contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave surface algorithm. A total of 34 contact pair surfaces were created (4 angle leg-to-web plate, 12 rivet head-to-angle leg, 12 rivet shank-to-angle leg hole surface and 6 rivet shank-to-web plate hole surface) considering Penalty friction formulation for tangential behaviour and default Hard Contact for normal behaviour. A coefficient of friction, $\mu$, equals to 0.3 was used in all the simulations.

Since no rivet clamping force was reported in [70], a relatively low clamping force value of 50 MPa and a fairly moderate clamping force value of 100 MPa were considered in the analyses. These values were selected to assess the predictive capability of the TCD in estimating the fatigue life as well as the pre-tension stress magnitude in the rivet shanks. In Abaqus, the clamping force was defined in the FE model at mid-length of the rivet shank using the ‘PRE-
Fatigue life evaluation of riveted built-up girders

TENSION SURFACE’ feature (BOLT LOAD) which applies a concentrated force over a user-defined cross-sectional surface.

The global model boundary condition and FE mesh remained unchanged in the shell-to-solid submodel analysis. The multistep shell-to-solid submodel FE analyses was carried out with the rivet clamping force application in the first step followed by subsequent steps where the results of the global shell model were interpolated onto the solid submodel.

7.3.3 Results and Discussions

Fatigue life estimation based on different formalisations of the TCD was carried out on results of the River Cree shell-to-solid submodel analyses with 50 and 100 MPa rivet clamping stresses. The $L$ vs. $N_f$ relationship determined for old structural metal (see Figure 3-4) was adopted to determine the critical length values required for fatigue life estimates based on the TCD method. Detail of the River Cree wrought-iron plain S-N curve used for the fatigue life estimation to the TCD is provided in Table 7-4 and plotted in Figure 7-13.

Moreover, fatigue life estimation based on the S-N approach was performed using all the S-N classifications presented in Figure 3-1, that is, ‘Class D’ [28], ‘Detail category 71’ [27], ‘WI-rivet Class’ [31] and ‘modified Class B’ [28], in conjunction with the nominal stresses obtained from the FE analysis results at the outmost fibre of the tension flange at the net cross-section.

Figures 7-17 (a) and (b) illustrate the TCD accuracy when employed to predict fatigue life of the River Cree built-up girders with 100 and 50 MPa rivet clamping forces, respectively. In order to create a comparison basis, the fatigue life estimations of the traditional S-N methods are also plotted in these figures. The results are expressed as experimental number of cycles to failure, $N_i$, vs. estimated number of cycles to failure, $N_{fe}$ with the experimental results CA scatter bands plotted in dashed lines.

The results in Figure 7-17 (a) shows that the TCD predicted fatigue lives that can be characterised with relatively high degree of nonconservatism especially in the medium-cycle fatigue regime. The predictions of the PM, LM, AM and VM were seen to be mostly outside CA scatter bands in ‘this’ region with fatigue strength predictions being between ±14%, ±23%, ±16% and ±19% error interval, respectively. The TCD predictions shown in Figure 7-17 (a) confirm that the rivet clamping forces less than 100 MPa may have been present in the girders tested by Burdon [70].
Figure 7-17 Comparison of the TCD and S-N method fatigue life estimation of the River Cree wrought-iron built-up girder under CA fatigue loading. (a) and (b) refer to fatigue life estimates for girders with 100 MPa and 50 MPa rivet clamping force respectively.
On the other hand, the results of the analysis on built-up girder model with 50 MPa rivet clamping force plotted in Figure 7-17 (b) is evidence that, majority of the TCD estimates are within the CA scatter bands in medium- and high-cycle fatigue regime. However, it can be also noted that reduction of the rivet clamping force to 50 MPa have caused the PM, AM, and VM predictions to fall inside the CA scatter bands but on the conservative side of the accuracy diagram. Therefore, it is reasonable to conclude that, based on the comparison of the TCD fatigue life predictions presented in the above accuracy diagrams, average rivet clamping forces between 50 to 100 MPa were present in the rivet shanks of the River Cree built-up girders. Another outcome worth noting here is that, the LM, as seen before, provided estimates with the least level of accuracy as compared to other TCD formalisations when applied to assess fatigue behaviour of riveted built-up girders.

On the contrary, the fatigue life predicted using the S-N methods seemed to be very poor and could be characterised by high degree of conservatism always falling mostly outside the CA scatter bands in the medium- or high-cycle fatigue regime. The predictions were noted to be sensitive to the detail classification adopted for fatigue assessment. The BS 5400-10 Class D gave the least conservative fatigue life predictions compared to other S-N classifications but still outside the experimental data CA scatter bands and on the conservative side.
Fatigue assessment of the Vindelälven River Bridge stringers

7.4.1 Experimental data

The Vindelälven riveted Railway Bridge was constructed in 1896 at Vännäsby in Sweden. The bridge consisted of three simply supported arch truss spans which was replaced in 1993 due to insufficient load-carrying capacity. Investigation of the design drawings revealed inadequate sway-bracing and braking stiffness members in the bridge structure because of poor detailing knowledge at the time of construction [48]. A total of 36 stringers and 9 floor beams were obtained from the old Vindelälven Bridge for full-scale fatigue testing at Chalmers University of Technology. Details of the stringers and cross-section dimensions are shown in Figure 7-18. Results of a series of tensile tests on flange material from the riveted girders revealed a mild steel material with an average yield stress, \( \sigma_y \), equal to 278 MPa, ultimate tensile stress, \( \sigma_{UTS} \), equal to 425 MPa, and a 35.2% elongation [48].

![Figure 7-18 Details of the Vindelälven Bridge mild steel riveted built-up girder (dimensions in mm)](image)

Akesson [46], in 1994, published results of a first series of tests conducted on nine of the stringers at a stress ratio, \( R \), between 0.14 to 0.28 with four different applied stress range levels of 40, 60, 80 and 100 MPa (stress ranges calculated at the outmost fibre of the tension flange based on net section area. Visual inspection of the selected stringers showed no signs of sever corrosion, no loose rivets and no visible fatigue cracks as a result of 100 years of service life. The CA fatigue test were carried out in four-point bending with 2 m gap between the static point loads. Total length of the stringers in most of the tests was 5 m and only in three tests it was reduced to 4 m to increase the flexural rigidity (tests at stress range levels of 40 and 60 MPa). Fatigue crack and failure occurred either at a rivet hole in the constant-
moment region or in majority of the cases at the tension flange-cross-diagonal connection (single or double L-profile riveted to stringer tension flange) shown as type I and type II in Figure 7-18. These wind-framing cross-bracing members were attached (riveted) to the tension flanges of the stringers in the original bridge structure. As shown in Figure 7-18, the location of rivets in these tension flange-cross-diagonal connections in the fatigue test set-up was at or near the constant-moment region and in the constant shear force region. Hence, these rivets were subjected to a certain amount of shear load as well as bending stresses close to the maximum bending stress magnitude, which made them prone to fatigue damage.

The average rivet clamping force in the rivet shanks of one of the stringers was estimated after full-scale CA fatigue test on that stringer was completed (test at stress range of 80 MPa). The average clamping stress in six rivets of the compression flange after 100 years of service life was estimated to be 151 MPa which was found to be 42% of the rivet material yield stress (i.e., average yield stress of 355 MPa) [48].

One of the primary conclusions of the experimental was that, the detail classification suggested by Eurocodes for fatigue assessment of riveted details (i.e., Detail Category 71) underestimates the fatigue life of riveted girders. The working stresses in the bridge superstructure was found to rarely exceed 42 MPa during the passage of the heaviest freight trains which is lower than the fatigue limit for riveted details taken as 52 MPa in structural steel code for riveted details [27]. Hence, accumulated fatigue damage in the Vindelälven Bridge can be assumed to be negligible [46].

In 1997, Kadir [58] performed another series of CA fatigue tests on three of the Vindelälven Bridge built-up stringers at stress range levels equal to 97 and 100 MPa. The results were seen to conform well with the results obtained in earlier full-scale tests by Akesson.

In 2001, Crocetti [79] published the results of another series of CA fatigue tests on five of the Vindelälven Bridge riveted built-up stringers performed at Chalmers University of Technology. The experiment was designed to investigate the existence of a constant amplitude fatigue limit (CAFL) for old steel riveted bridge members. The stringers were tested at stress range levels of 40 and 60 MPa initially up to 10 to 20 million cycles and consequently at a much higher stress range (about 100 MPa) to complete failure. The test results were then compared to the results of previous tests performed on the Vindelälven Bridge stringers at Chalmers University of Technology. The study concluded that the CAFL suggested in Eurocodes Detail Category 71 (CAFL = 52 MPa) [27] is applicable for old steel riveted bridges.
Fatigue life evaluation of riveted built-up girders

The final series of CA fatigue tests on three of the Vindelälven Bridge built-up stringers was carried out in 2002 by Al-Emrani [48] at stress range levels of 60 to 100 MPa [12]. These tests were performed to assess the efficiency of drilling stop hole as a temporary arrest for propagating fatigue cracks. The technique was found to be effective in arresting the crack development and delaying the propagation by up to 215,000 loading cycles.

Summary of all the full-scale fatigue tests carried out at Chalmers University of Technology on the Vindelälven Railway Bridge built-up stringers is reported in Table 7-5.

The above fatigue test results and the correspondence S-N curve together with the 10% and 90% CA scatter bands representing the scatter in the test data are plotted in Figure 7-19.

Table 7-5  Summary of the experimental test results on the Vindelälven Bridge stringers under static and CA fatigue loading [12]

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>R</th>
<th>m</th>
<th>UTS (MPa)</th>
<th>Δσ (^a) (MPa)</th>
<th>(T_σ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain</td>
<td>-</td>
<td>-</td>
<td>4</td>
<td>-</td>
<td>217.4</td>
<td>1.467</td>
<td></td>
</tr>
<tr>
<td>Built-up girders</td>
<td>20</td>
<td>20</td>
<td>0.14-0.28</td>
<td>3.38</td>
<td>425</td>
<td>139.6</td>
<td>1.915</td>
</tr>
</tbody>
</table>

\(^a\) Fatigue strength at \(N_f = 2 \times 10^6\) cycles to failure related to net section area (see Figure 7-18). Plain S-N curve refers to BS5400-10 Class B mean S-N curve [28]

The CA scatter bands for S-N data of the built-up girders shown by dashed lines in Figure 7-19 refer to the 90% and 10% probabilities of survival, \(P_σ\), determined under the hypothesis that \(\log N\) is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. Dispersion ratio of stress amplitude, \(T_σ\), (calculated for stress amplitude at \(2 \times 10^6\) cycles to failure) for plain specimen S-N curve equals to 1.467 and for the Vindelälven Bridge built-up stringer S-N curve to 1.915. The statistical dispersion of fatigue results obtained for the built-up stringers is slightly larger but comparable to that of plan material.

The statistical analysis of the Vindelälven Bridge built-up stringer fatigue data to estimate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM [71]. The fatigue data expressed as ‘run-out’ were not used in the process of S-N curve estimation but plotted in Figure 7-19. A single-slope S-N curve was considered to be consistence with the previous case studies investigated in this thesis.
Fatigue life evaluation of riveted built-up girders

The results of the CA fatigue test on the Vindelälven Bridge built-up stringers [12]. The stringer S-N data refer to the gross cross-section area (see Figure 7-18). Plain S-N curve refers to the Class B mean S-N curve obtained from [28].

It should be added that because of non-existence of a plain material S-N data for the Vindelälven Bridge stringers, the mean S-N curve Class B [28] which corresponds to mild steel plain material is used in this study for fatigue life estimation according to the TCD method. Detail of this curve is reported in the Table 7-5 and plotted in Figure 7-19.

### 7.4.2 FE analysis of the Vindelälven Bridge built-up stringer

Shell-to-solid submodelling technique in Abaqus/CAE v6.10 was used to model and analyse the Vindelälven Bridge built-up stringer. For simplicity of the analysis, the tension flange-cross-diagonal connections shown as type I and type II in Figure 7-18 were not included in the modelling. Hence, the neck rivet in the web plate-tension flange connection in the constant-moment region was expected to be the critical hot-spot location which was also the location of fatigue failure in one of the tests performed by Akesson [46]. Effects of such simplification on the estimated fatigue life values was investigated and discussed later in this section.
Fatigue life evaluation of riveted built-up girders

The initial shell global model was created by transforming the cross-section in Figure 7-18 to an equivalent I-section with the same second moment of inertia and total depth without considering the area reduction due to rivet holes. The above I-section consisted of an equal size 238-mm wide, 5.5-mm thick compression and tension flange and an 819-mm deep, 25.89-mm thick web plate. Linear elastic behaviour was assumed for the mild steel material during the analysis with Young’s Modulus and Poisson’s ratio values of 200 GPa and 0.3, respectively. The global model was meshed using 8-noded shell elements (S8R) as shown in Figure 7-20. The global model FE mesh consisted of about 1130 S8R type elements. As to the boundary condition, the tension flange of the global model was constrained against translation in the X-, Y- and Z-directions at one end and in the X- and Y- directions at the other end along the shell edge.

In a multistep analysis, the global model was subjected to four-point bending at the locations illustrated in Figure 7-18. The loading was applied as static point loads to generate stress values between 50 to 180 MPa in the outmost fibre of the tension flange in the constant-moment region. The resultant output file was used in the consequent step of the analysis to drive the shell-to-solid submodel. The solid submodel superimposed on the shell global model is shown in Figure 7-21 which was created by cutting and removing a length of 340 mm of the central section of the global model and replacing it with solid submodel of the stringer. The submodel consisted of two tension and two compression flange angles, a web plate and six rivets with the dimensions specified in Figure 7-18. Brick elements (C3D20) were used to mesh the solid submodel. 48 elements were used at the perimeter of the rivets/rivet holes in tension flanges, which were expected to be the fatigue-critical hot-spots. Following the modelling recommendations mentioned in the previous chapters, seven elements were used through the thickness of the vertical legs of the tension flanges in the submodel to provide adequate stress readout points near the rivet hole edge. Detail of the FE mesh used in the solid submodel is shown in a close-up image in Figure 7-22.

The nodes along the edges of the global shell model which were formed after cutting and removing the 340-mm central section were used as driving nodes to interpolate the results of the shell global model to the solid submodel. A default value of 10% of the maximum shell thickness (in this case equal to 2.6 mm) was used as the centre zone size.
Figure 7-20  FE mesh used to model the shell global model of the Vindelälven Bridge stringer

Figure 7-21  Close-up image of the solid submodel of the Vindelälven Bridge stringer overlaid on the shell global model
Fatigue life evaluation of riveted built-up girders

Figure 7-22 Close-up of the FE mesh used to model the solid submodel. (Approximately 132,000 C3D20 brick elements)

The clearance between the rivet shank and the hole surface in the submodel was assumed to be equal to zero. Contact surfaces were modelled between the contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave surface algorithm. A total of 34 contact pairs were defined (4 angle leg-to-web plate surface, 12 rivet head-to-angle leg surface, 12 rivet shank-to-angle leg hole surface and 6 rivet shank-to-web plate hole surface) considering Penalty friction formulation for tangential behaviour and default Hard Contact for normal behaviour. A coefficient of friction, $\mu$, equals to 0.3 was used in all simulations.

Rivet clamping force value of 151 MPa was considered for the analysis. This is equal to the average rivet clamping force determined for six rivets in compression flange of one of the Vindelälven Bridge stringers [46]. In the FE model, the clamping force was defined at mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus which applies a concentrated force over a user-defined cross-sectional surface.

In the shell-to-solid submodel analysis, the boundary condition and FE mesh of the global shell model remained unchanged. The multistep shell-to-solid FE analysis was carried out with
application of only the clamping force in the first step followed by the subsequent steps where results of the global shell analysis were interpolated onto the solid submodel.

7.4.3 Results and Discussions

All formalisations of the TCD method were employed to estimate fatigue life of the Vindelälven Bridge stringer by post processing the results of the shell-to-solid analysis. The \( L \) vs. \( N_f \) relationship for old structural metal determined in previous chapters was adopted to calculate the critical length values required for life estimation based on the TCD. The Class B mean S-N curve plotted in Figure 7-19 (representative of mild steel plain material) was used for the purpose of fatigue life prediction to the TCD.

Additionally, three S-N classifications proposed in different fatigue design and assessment codes of practice for riveted structures, (i.e., mean S-N curves of ‘Class D’ [28], ‘Detail category 71’ [27], and ‘modified Class B’ [27]), were used in conjunction with the nominal stresses obtained from the FE analysis results at the outmost fibre of the tension flange at the net cross-section to predict fatigue life of the stringers subjected to various stress range levels.

The results are presented in the form of experimental number of cycles to failure, \( N_i \), vs. estimated number of cycles to failure, \( N_{f,e} \) with the experimental data CA scatter bands plotted using dashed lines to indicate the accuracy of the predictions. The TCD method accuracy in estimating fatigue life of mild steel built-up stringer is shown in the reported diagram in Figure 7-23. The above results correspond to stringers with 151 MPa rivet clamping forces. The fatigue-critical location was considered to be the rivet hole edge in the tension flange-web plate interface (neck rivet hole in the tension flanges).

It is evident that the TCD was very successful in predicting fatigue lives always falling within the scatter bands of the experimental data. However, the TCD predictions presented in Figure 7-23 (a) appears to be slightly on the nonconservative side. In more detail, different formalisations of the TCD, (i.e., the PM, LM, AM and VM), estimated fatigue strength values with maximum error interval between -4% to -13% compared to the experimental data. It can also be noted that, the PM, AM and VM estimates were effectively identical while the LM results, as previously seen, contained slight degree of nonconservatism as compared to the other TCD formalisations but still within the CA scatter bands.
Fatigue life evaluation of riveted built-up girders

Figure 7-23 Comparison of the TCD and S-N method fatigue life predictions of the Vindelälven Bridge mild steel built-up stringer under four-point bending (a) refers to fatigue life at the neck rivet location in the tension flange (b) refers to fatigue life at the tension flange-cross-bracing location (shown as Type II in Figure 7-18) calculated by modifying the fatigue strength results obtained in (a)
Fatigue life evaluation of riveted built-up girders

However, the level of nonconservatism in the above fatigue life predictions can be explained by comparing the difference between the net section nominal stress range values at the neck rivet level to corresponding value at the level of the rivet hole in tension flange-cross-bracing connection (shown as Type I or II in Figure 7-18). The obtained net section nominal stress range at the location I or II is about 9.6% larger compared to the corresponding value at the neck rivet (the neck rivet lower edge is 30 mm above the tension flange outermost fibre); therefore, increasing the fatigue strength results obtained for the ‘neck rivet’ by 9.6% could represent a reasonable estimate for potential fatigue strength values for locations I or II for the same applied loads (hereafter called modified fatigue strength). Figure 7-23 (b) shows the TCD method accuracy when the modified fatigue strength values are used to predict the number of cycles to failure for the built-up stringer. Results in Figure 7-23 (b) conforms more closely with the experimental data obtained for Vindelälven Bridge built-up stringer reported in [7, 43, 76]. This is because majority of the fatigue failures reported in the experimental investigations on the Vindelälven Bridge built-up stringer occurred at the rivet hole edge in location II [46].

The TCD predictions in Figure 7-23 (a) and (b) verify that the magnitude of the average rivet clamping force value of 151 MPa determined by Akesson [46] for the Vindelälven Bridge stringer rivets to be a good estimate. This finding once more confirms the predictive capabilities of the TCD method in fatigue assessment of full-scale riveted bridge members.

Regarding the fatigue life predictions based on the S-N method, majority of the predictions were seen to be inside the CA scatter bands in the medium- and high-cycle fatigue regime. The predictions of Class D were found to be slightly on the conservative side specially in the medium-cycle regime. The predictions of Detail category 71 could be said to be in line with that of the TCD method. In the case of full-scale riveted girders loaded in three- or four-point bending, the S-N method fatigue life predictions are expected to be reasonably accurate since the S-N classes suggested in most codes of practice are derived from the experimental data obtained for such full-scale riveted bridge members. However, since the experimental data used to estimate the S-N curve of the detail classes suggested for old riveted structures consisted of full-scale members with variable degree of corrosion, defects and variable rivet clamping stresses, the S-N approach fatigue estimates is seen to have a certain degree of conservatism in some of the cases.
Fatigue life evaluation of riveted built-up girders

7.5 Fatigue assessment of wrought-iron built-up girders

In 1990, Mang and Bucak [49, 77, 78] published results of a series of full-scale fatigue tests on a section of the Blumberg Bridge. The Blumberg Bridge was built in 1887 and was part of the museum railway in the south of Germany.

The CA fatigue test on a section of the Blumberg Bridge was performed at the stress level of 137 MPa calculated at the outer surface of the main girder tension chord at a stress ratio of, $R = 0.1$ [80]. After the test, the Blumberg bridge was then cut into its constituent members to perform fatigue tests on the main and longitudinal girders individually [80].

Two of the Blumberg bridge main girders with the configurations and arrangements illustrated in Figure 7-24 were tested in four-point bending at a stress ratio, $R$, equal to +0.2. The main girders were subjected to a stress range of 135 MPa (calculated at the outmost fibre of main girder tension flange) up to $10^7$ stress cycles and then since no failure occurred, the stress was increased to 165 MPa. Unbroken parts of the two tested main girders of the Blumberg bridge were then cut to create three additional 1500-mm long test specimens. These girders were tested in three-point bending at a stress ratio, $R$, equal to +0.1 and +0.35 to investigate the effects of the mean stress.

Four further 1500-mm long test specimens were prepared from the longitudinal girders of the Blumberg Bridge and tested in three-point bending at a stress ratio, $R$, equal to +0.1.

![Figure 7-24 Details of the Blumberg Bridge riveted wrought-iron built-up main girder (dimensions in mm) [81]. A 40-mm vertical distance was assumed for flange neck rivet hole centreline to outer surface of the horizontal angle leg due to the lack of data](image-url)
Mang and Bucak also performed fatigue tests on full-scale riveted built-up girders from the Stahringen Bridge (built in 1895) [81]. These tests were carried out on 3300 mm long girders in four-point bending with identical support and loading conditions as shown in Figure 7-24 and at a stress ratio, $R = +0.2$. The results from the full-scale fatigue tests conducted by Mang and Bucak on riveted girders are summarised in Table 7-6 and plotted in Figure 7-25. The results in Figure 7-25 also include result of a fatigue test on the main girder of the Bruchsal Bridge at a similar stress ratio (a suburban train bridge in Berlin) [81]. The plain S-N data presented in Table 7-6 and Figure 7-25 corresponds to the a structural wrought-iron material taken from [54] obtained at a stress ratio, $R$, equal to +0.1. This plain material S-N curve was used in this case study for fatigue life prediction based on the TCD method.

Investigation by Mang and Bucak on old steel bridges [81] also included a series of tensile tests on wrought-iron specimens obtained from various parts of three different bridges dating from 1877-1899 (including the Blumberg and the Stahringen bridge). The results of the tensile tests revealed an average yield stress, $\sigma_y$, equal to 284 MPa, an ultimate tensile stress, $\sigma_{UTS}$, equal to 390 MPa, a Young’s Modulus, $E$, equal to 190 GPa and a 24% Elongation.

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>$R$</th>
<th>$m$</th>
<th>UTS (MPa)</th>
<th>$\Delta\sigma$ a (MPa)</th>
<th>$T_\sigma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain b</td>
<td>14</td>
<td>-</td>
<td>0.1</td>
<td>6.84</td>
<td>NA</td>
<td>193.6</td>
<td>1.356</td>
</tr>
<tr>
<td>Built-up girders</td>
<td>14</td>
<td>20</td>
<td>0.1-0.35</td>
<td>3.6</td>
<td>390</td>
<td>120.4</td>
<td>1.239</td>
</tr>
</tbody>
</table>

a Fatigue strength at $N_f = 2 \times 10^6$ cycles to failure related to net section area calculated at the outmost fibre of the tension chord (see Figure 7-24). b Refers to structural wrought-iron material plain S-N curve taken from [54]

The statistical analysis of the fatigue data of built-up girders (presented in Figure 7-25) to approximate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM [71]. The fatigue data expressed as ‘run-out’ were not used in the process of S-N curve estimation but plotted in Figure 7-25. A single-slope S-N curve was considered to be consistence with the previous case studies investigated in this thesis.

The CA scatter bands for the S-N data, shown by dashed lines in Figure 7-25, refer to the 90% and 10% probabilities of survival, $P_S$, determined under the hypothesis that $Log N$ is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. Dispersion ratio of the stress range, $T_\sigma$, (calculated for stress
ranges at $2 \times 10^6$ cycles to failure) for the plain specimen S-N curve shown in Figure 7-25 equals to 1.379 and for the built-up girder S-N data equals to 2.434. The statistical dispersion of fatigue test results obtained for the built-up girders is considerably larger compared to that of the plan material S-N data.

![Figure 7-25](image)

**Figure 7-25** The results of the CA fatigue tests on the full-scale girders of the Blumberg bridge (museum railway), the Stahringen bridge and the Bruchsal bridge [81]. The stringer S-N data refers to the gross cross-section area (see Figure 7-24). The plain S-N curve refers to a structural wrought-iron material S-N curve obtained from [54]

It should be noted that, results of three-point bending fatigue tests on the remaining parts of the Blumberg bridge main girder are slightly towards the lower CA scatter band which could be caused by the previous fatigue loading endured during the four-point bending tests on the main girder. This could also explain the larger $T_\sigma$ value for the built-up girders’ S-N data.
7.5.1 FE analysis of Blumberg bridge built-up main girder

Abaqus/CAE v6.10 shell-to-solid submodelling technique was adopted to model and analyse the Blumberg bridge built-up girder. Because of the lack of information regarding the Stahringen and Bruchsal bridge girders’ cross-section detail, only the simulation for four- and three-point bending fatigue tests on the Blumberg bridge main girder was performed.

For this purpose, the initial shell global models (3300-mm and 1500-mm long girders in four- and three-point bending loading, respectively) were created by transforming the cross-section shown in Figure 7-24 to an equivalent I-section with the same second moment of inertia and total depth without considering the area reduction because of the rivet holes. The above equivalent I-section consisted of a 315.227-mm wide, 20-mm thick equal size compression and tension flanges and a 393-mm deep, 24-mm thick web plate. A linear elastic behaviour was assumed for the mild steel material during the FE analysis with a Young’s Modulus and Poisson’s ratio values of 190 GPa and 0.3, respectively. The global models were meshed using 8-noded shell elements (S8R) as shown in Figure 7-26. The 3300 mm and 1500 mm global model FE meshes consisted of about 1130 and 1,050 S8R type elements, respectively. As to the boundary condition, the tension flanges of the global model was constrained against translation in the X-, Y- and Z-directions at one end and in the X- and Y-directions at the other end along the shell edge.

In a multistep analysis, the global models were subjected to four-point bending (at locations shown in Figure 7-24) or to three-point bending (a central point load). The loading was applied as static point load(s). In case of the main girder subjected to four-point bending, the load levels were selected as such to result in stress values ranging between 51 to 179 MPa in the outmost fibre of the tension cover plate in the constant-moment region. In the case of three-point bending analysis, the corresponding load levels were chosen as such to create maximum stress values ranging between 61 to 120 MPa in the outmost fibre of the tension cover plate at the centre. The resultant output files were used in the subsequent step of the FE analyses to drive the shell-to-solid submodels.

The solid submodel superimposed on the shell global model is shown in Figures 7-26 and 7-27 which was created by removing a 300-mm length of the global models’ central section and replacing it with the solid submodel of the main girder.
The symmetric solid submodel consisted of two tension and two compression flange angles, two tension and two compression cover plates, a web plate and eight rivets with the dimensions specified in Figure 7-24. Full-integration brick elements (C3D20) were used to mesh the solid submodel. 40 elements were used at the perimeter of the rivet holes in tension cover plates, which was expected to be the fatigue-critical locations. Following the modelling recommendations outlined in the previous chapters, 10 elements were used through the thickness of the outer tension cover plate of the main girder (10 mm thick plate) in an attempt to form adequate stress readout points near the rivet hole edge. Details of the FE mesh used to model the solid submodel are shown in a close-up in Figure 7-29.

The nodes along the edges of the global shell model which were fashioned after cutting and removing the 300-mm central section were used as driving nodes to interpolate the results of shell global model to the solid submodel. The default value of 10% of the maximum shell thickness (in this case equal to 2.4 mm) was used as the centre zone size.

As to the submodel modelling, the clearance between the rivet shank and the hole surface was assumed to be equal to zero. Contact surfaces were modelled between the contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave surface algorithm.
Fatigue life evaluation of riveted built-up girders

Figure 7-27  The FE solid submodel with a close-up of the central section for the 3300 mm long main girder of the Blumberg bridge overlaid on the shell global model analysed under four-point bending

Figure 7-28  The FE solid submodel of the Blumberg bridge 1500 mm long main girder overlaid on the shell global model and analysed under three-point bending
A total of 52 contact pair surfaces were defined (2 cover plate-to-cover plate, 4 angle leg-to-cover plate, 4 angle leg-to-web plate, 2 web plate-to-cover plate, 12 rivet head-to-angle leg, 4 rivet head-to-cover plate, 4 rivet shank-to-web plate hole surface, 8 rivet shank-to-cover plate hole surface and 12 rivet shank-to-angle hole surface) considering Penalty friction formulation for tangential behaviour and default Hard Contact for normal behaviour. A coefficient of friction, $\mu$, equals to 0.3 was used in all simulations.

In the investigation by Mang and Bucak on fatigue assessment of old steel bridge members [49, 77, 78], no attempt was made to determine the average rivet clamping force in the rivets of the tested girders. Hence, in this study, to assess the predictive capability of the TCD method in estimating the pre-tension stress magnitude in the rivets as well as fatigue life of full-scale riveted girders, a relatively low clamping force value of 50 MPa and a fairly moderate clamping force value of 100 MPa were considered in the analyses. In the FE model, the clamping force was defined at the mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus which applies a concentrated force over a user-defined cross-sectional surface. The boundary conditions and FE mesh for the global shell models in the shell-to-solid submodel analyses remained unchanged.
Fatigue life evaluation of riveted built-up girders

The multistep shell-to-solid analyses were carried out with application of only the rivet clamping force in the first step followed by subsequent steps where the results of the global shell model were interpolated onto the solid submodel. The effects of the applied rivet clamping force were set to propagate to the next steps of the analysis.

7.5.2 Results and Discussions

The TCD method in the form of the PM, LM, AM and VM was used to predict fatigue life of the Blumberg bridge girders by post processing the results of the four- and three-point bending shell-to-solid analyses. In order to calculate the critical length values required for fatigue life prediction based on the TCD, the $L$ vs. $N_f$ relationship for old structural metal calibrated in the previous chapters was adopted. It should be said here that plain S-N data for a structural wrought-iron material at a stress ratio greater than +0.1 was not found in the technical literature. Therefore, since material-specific plain S-N data for the Blumberg bridge wrought-iron was not available, a plain S-N curve for structural wrought-iron obtained from a different source [54] determined at a stress ratio equal to +0.1 was adopted in fatigue life prediction to the TCD method. The detail of this plain S-N curve is shown in Figure 7-25.

Additionally, four commonly used S-N classifications proposed in fatigue design and assessment codes of practice for riveted structures (i.e., mean S-N curves of ‘Class D’, ‘Detail category 71’, ‘W1-rivet Class’ and ‘modified Class B’), were used in conjunction with the nominal stresses obtained from the FE analysis results at the outermost fibre of the tension cover plate at the net cross-section to predict fatigue life of the built-up girder subjected to various stress range levels.

Figure 7-30 illustrates the TCD method accuracy when employed to predict fatigue life of the Blumberg bridge girders with 50 and 100 MPa rivet clamping force under three- and four-point bending fatigue loading. The fatigue life estimates based on the traditional S-N method are also plotted in the above figure to create a comparison basis. The results are presented as experimental number of cycles to failure, $N_i$, vs. estimated number of cycles to failure, $N_{i,e}$ with dashed lines representing the CA scatter bands of the experimental data.

The results in Figures 7-30 (a) and 7-30 (b) refer to fatigue life predictions for the Blumberg bridge 3300 mm long main girder under four-point bending with 50 and 100 MPa rivet clamping force, respectively.
Fatigue life evaluation of riveted built-up girders

Figure 7-30  Comparison of the TCD and S-N method fatigue life predictions for the Blumberg bridge wrought-iron built-up girders under four- and three-point bending with stress ratio, $R$, varying between +0.1 to +0.35. (a) and (b) refer to fatigue life predictions for 3300-mm long main girder under four-point bending with 50 and 100 MPa rivet clamping force, respectively. (c) refers to fatigue life predictions for 1500-mm long girder under three-point bending with 100 MPa rivet clamping force.
Fatigue life estimates reported in Figures 7-30 (a) and (b) show a good level of accuracy in predictions of the TCD method with majority of predictions in medium- and high-cycle fatigue regime falling within the experimental data CA scatter bands. However, direct comparison of the above figures reveal that the clamping forces present in the rivets connecting the angles to girders’ web in the tested specimens were in the order of magnitude of about 100 MPa for the Blumberg Bridge main girders tested by Mang and Bucak [49,77,78]. Further confirmation was provided by the fatigue strength predictions of all different formalisations of the TCD method given in Figure 7-30 (b) which were seen to be within an error factor of ±20%.

The results in Figure 7-30 (c) refer to fatigue life predictions for remains of the Blumberg bridge main girders tested under three-point bending (1500-mm long built-up girders) with 100 MPa rivet clamping force. These girders were experimentally tested at a stress ratio, $R$, between +0.1 to +0.3. It is evident that the TCD was very successful in predicting fatigue lives always falling within the experimental data CA scatter bands. The PM, LM, AM and VM estimated fatigue strength with maximum error interval of ±7%, ±12%, ±6% and ±6%, respectively in the medium- and high-cycle fatigue regime. This confirms a high degree of accuracy achieved in the predictions of the TCD method. It also can be noted that predictions made by the AM and the VM are effectively similar in the medium- and high-cycle fatigue regime.

In overall, very high level of accuracy was found in the TCD method predictions which is encouraging, since, from statistical point of view, a predictive method cannot be expected to be more accurate than the actual experimental data. This was true despite the fact that the Blumberg Bridge wrought-iron plain S-N data for stress ratios equal to 0.2 or 0.35 was not available to incorporate in fatigue life predictions based on the TCD. Therefore, it could be presumed that the slight deviation of the TCD predictions from the best fit line may be improved if such relevant or equivalent plain material S-N data was used.

In contrast, the fatigue life predictive capability of the S-N method was found to suffer with some degree of conservatism. Only Class D and Detail category 71 were capable of making predictions falling within the CA scatter bands. Class D was seen capable making very accurate predictions comparable to the TCD method predictions. The estimates of Detail category 71, however, appeared to increase in degree of conservatism towards the high-cycle regime.
7.6 Fatigue assessment of Hinterrhein Bridge tapered built-up girders

In 1999, Bassetti et al. [60] published results of a series of CA fatigue tests on full-scale built-up riveted girders. Six cross-girders were obtained after the replacement and dismantling of an old railway bridge in 1993. The bridge over the Hinterrhein river, in Thun (Switzerland), was constructed in 1901 and been in service since 1902. The configurations and dimensions of the Hinterrhein Bridge cross-girders are shown in Figure 7-31. The tapered cross-girder had a 1260-mm long central section and two 1665 mm tapered side sections. Different elements of the tension and compression chord (angles and cover plate) were connected to the web plate and to each other using 24-mm diameter rivets with 126 and 119 mm rivet spacings in the central and tapered side sections, respectively. The tensile tests on six samples obtained from tension angles of the cross-girders confirmed the material of the built-up girders as mild steel with an average yield stress, $\sigma_y$, equal to 302 MPa, an ultimate tensile stress, $\sigma_{UTS}$, equal to 412 MPa and a Young’s Modulus, $E$, equal to 212 GPa [60].

Bassetti et al. [60] aimed to assess the effectiveness of using pretensioned or non-pretensioned carbon fibre reinforced plastic strips (CFRP-strip) bonded to the fatigue-critical section of the old riveted members as a repair method compared to more conventional repair techniques (i.e., replacing rivets with high-strength bolts, drilling a stop hole at the fatigue crack tip and adding cover plate to the cracked section). The investigation concluded that the conventional repair techniques are local remedies and mostly effective when the fatigue crack is detected in an early phase and is short in length. The study also experimentally examined the effects on fatigue strength of non-pretensioned CFRP-strip bonded to small-
scale steel specimen which resulted in slower fatigue crack growth rates and an increase of fatigue life by a factor of 3 compared to steel alone specimen.

Another main aim of the experiments carried out by Bassetti et al. [60] was to investigate the fatigue resistance of old riveted bridge members. Full-scale fatigue tests were performed on four of the Hinterrhein Bridge cross-girders under four-point bending with the support and loading configurations as illustrated in Figure 7-31. The CA fatigue tests were conducted at a stress ratio, $R$, equal to +0.1. For three of the cross-girders, upon detection of a fatigue crack, conventional techniques (i.e., replacing the rivet at which crack was initiated with high-strength bolt, drilling a stop hole at the crack tip, or adding a cover plate ‘patch’ to the cracked section) were used to repair the section and the test was continued until further cracks at a different rivet location was detected. This allowed for more than one fatigue test result to be obtained from the tests on each specific cross-girder. Fatigue cracks were formed exclusively at a tension flange rivet hole either in the constant-moment region (central section) or very close to this region in the maximum shear load section (tapered section) of the cross-girder.

A summary of the fatigue test results by Bassetti et al. on tapered cross-girders of the Hinterrhein Bridge are shown in Table 7-7 and plotted in Figure 7-32. The results of the previous CA fatigue tests on the full-scale mild steel HEB 1000 rolled girders with riveted cover plates carried out by Bruhwiler et al. [41] are also plotted in this figure for comparison.

**Table 7-7 Summary of the experimental test results on mild steel built-up cross-girders by Bassetti et al. [60]**

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>No. of data</th>
<th>Hole diameter (mm)</th>
<th>$R$</th>
<th>$m$</th>
<th>UTS (MPa)</th>
<th>$\Delta \sigma$ (MPa)</th>
<th>$T_\sigma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain $^b$</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4</td>
<td>NA</td>
<td>184.9</td>
<td>1.467</td>
</tr>
<tr>
<td>Built-up girder</td>
<td>13</td>
<td>24</td>
<td>0.1</td>
<td>3</td>
<td>412</td>
<td>116.9</td>
<td>1.842</td>
</tr>
</tbody>
</table>

$^a$ Fatigue strength at $N_f = 2 \times 10^6$ cycles to failure related to net section area calculated at the outmost fibre of the tension chord (see Figure 7-31). $^b$ Refers to BS 5400-10 Class B mean S-N curve obtained from [28]
It was concluded by Bassetti et al. [60] that the results of the experimental fatigue tests on the Hinterrhein Bridge built-up cross-girder conforms well with the results of previous tests performed at ICOM by Bruhwiler et al. [41]. Furthermore, measurements of the pretension in two of the rivets from tapered cross-girders determined rivet clamping forces of about 140 and 160 MPa which confirms presence of relatively high rivet clamping force values similar to those found in the mild steel HEB 1000 rolled girders with riveted cover plate tested in [41]. The statistical analysis of the fatigue data of the Hinterrhein Bridge built-up cross-girder to estimate the mean S-N relationship and the CA scatter bands was performed according to the procedure described by ASTM in [71]. A single-slope S-N curve was considered to be consistence with the previous case studies.

The CA scatter bands for built-up girder S-N data shown by dashed lines in Figure 7-32 refer to the 90% and 10% probabilities of survival, $P_S$, determined under the hypothesis that $\log N$...
is normally distributed (Log-Normal distribution of number of cycles to failure for each stress level) assuming a 95% confidence level. Dispersion ratio of the stress range, $T_o$, (calculated for the stress ranges at $2 \times 10^6$ cycles to failure) for the plain specimens and the riveted built-up cross-girder S-N curves equals to 1.467 and 1.842, respectively. The statistical dispersion of fatigue results obtained for the built-up cross-girders is slightly larger but comparable to that of the plan specimen.

7.6.1 FE analysis of Hinterrein Bridge built-up cross-girder

Abaqus/CAE v6.10 shell-to-solid submodelling technique was adopted to model and analyse the Hinterrein Bridge cross-girders. In order to create the shell global model, the central and end cross-sections of the tapered cross-girder shown in Figure 7-31 were transformed into two equivalent I-sections with the same second moment of inertia, total depth and neutral-axis position without considering the area reduction because of the rivet holes. The dimensions of the I-sections representing the central and end cross-sections of the shell global model are shown in Figure 7-33.

![Figure 7-33 Equivalent I-sections used to model the global shell model of the Hinterrein Bridge built-up cross-girder (mm)](image)

The global model was build up in three stages; initially a central section of 1260 mm in length was created by extruding the associated equivalent I-section shown in Figure 7-33. In the second stage, side sections, with 1665 mm length each were modelled by joining individually
generated tension and compression flanges and a tapered web plate. Finally, the modelled central and side sections were assembled and connected together using the ‘Merge Instances’ command in the Abaqus Assembly tab.

A linear elastic behaviour was assumed for the mild steel material during the analysis with a Young’s Modulus and Poisson’s ratio values of 212 GPa and 0.3, respectively. The global model was meshed using 8-noded shell elements (S8R) as shown in Figure 7-34. The global model FE mesh consisted of about 2300 S8R type elements. Regarding to the boundary condition, the tension flange of the global model was constrained against translation in the X-, Y- and Z-directions at one end and in the X- and Y- directions at the other end along the shell edge.

In a multistep analysis, the global model was subjected to four-point bending using the support and loading configurations shown in Figure 7-31. The load magnitudes were selected as such to generate stress values ranging between 48 to 237 MPa in the outmost fibre of the tension flange in the constant bending moment region (central section). The resultant output file was used in the subsequent step of the analysis to drive the shell-to-solid local submodel.

![FE mesh used to model the Hinterhein Bridge built-up cross-girder shell global model](image)

The solid submodel superimposed on the shell global model is shown in Figure 7-35 which was created by removing a 378 mm length of the central section of the global model and
replacing it with the solid model of the cross-girder. The submodel consisted of two tension and two compression flanges, a compression cover plate, a web plate and 12 rivets (24 mm in diameter) with the dimensions specified in Figure 7-31. Full-integration brick elements (C3D20) were used to mesh the solid submodel. 44 elements were used at the perimeter of the rivet holes in tension flanges, which is expected to be the fatigue-critical locations. Following the modelling recommendations given in the previous chapters, seven elements were used through the thickness of the tension flange vertical leg (the leg in contact with the web plate). Detail of the FE mesh used in the submodel is presented in a close-up image shown in Figure 7-36.

*Figure 7-35 The FE solid submodel and a close-up of the central section of the Hinterrhein Bridge built-up cross-girder overlaid on the shell global model*
The nodes along the edges of the global shell model which were formed after cutting and removing the 378-mm central section were used as driving nodes to interpolate the results of shell global model to the solid submodel. A default value of 10% of the maximum shell thickness (in this case equal to 2.2 mm) was used as the centre zone size.

With regard to the submodel modelling, the clearance between the rivet shank and the hole surface was assumed to be equal to zero. Contact surfaces were modelled between the contacting bodies using standard Surface-to-Surface contact method with finite sliding and master-slave surface algorithm. A total of 61 contact pair surfaces were defined (4 angle leg-to-web plate surface, 2 angle leg-to-cover plate surface, 1 cover plate-to-web plate surface, 18 rivet head-to-angle leg surface, 6 rivet head-to-cover plate surface, 18 rivet shank-to-angle leg hole surface, 6 rivet shank-to-web plate hole surface and 6 rivet shank-to-cover plate hole surface) considering Penalty friction formulation for tangential behaviour and default Hard Contact for normal behaviour. A coefficient of friction, $\mu$, equals to 0.3 was used in all the simulations.
A rivet clamping force value of 140 MPa was considered for the analysis as was experimentally determined by Bassetti et al. [60]. This is equal to the lower rivet clamping force value determined for 2 of the rivets in one of the cross-girders. In the FE model, the clamping force was defined at the mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ feature (BOLT LOAD) in Abaqus which applies a concentrated force over a user-defined cross-sectional surface.

In the shell-to-solid analysis, the boundary condition and FE mesh for the global shell model remained unchanged. The multistep shell-to-solid submodel FE analysis was carried out with application of only the clamping force in the first step followed by the subsequent steps where the result of the global shell model was interpolated onto the solid submodel.

7.6.2 Results and Discussions

All different formalisations of the TCD method were used to estimate fatigue life of the Hinterrhein Bridge tapered cross-girder by post processing the results of the shell-to-solid submodel. The $L$ vs. $N_f$ relationship for old structural metal determined in the previous chapters was adopted to calculate the critical length values required for fatigue life estimation based on the TCD method. Since the plain material S-N data specific to the Hinterrhein Bridge mild steel was not available, Class B mean S-N curve [28] plotted in Figure 7-32 (representative of mild steel plain material) was used for the purpose of fatigue life prediction to the TCD.

Additionally, three commonly used S-N classifications proposed in fatigue design and assessment codes of practice for riveted structures (i.e., mean S-N curves of ‘Class D’, ‘Detail category 71’, and ‘modified Class B’), were used in conjunction with the nominal stresses obtained from the FE analysis results at the outmost fibre of the tension flange at the net cross-section to predict fatigue life of the tapered cross-girder subjected to various stress range levels.

Figure 7-37 illustrates the accuracy of the TCD method when employed to predict fatigue life of the Hinterrhein Bridge mild steel tapered cross-girder with 140 MPa rivet clamping force under four-point bending fatigue loading. The fatigue life prediction based on the traditional S-N method is also plotted in this figure to create a comparison basis. The results are presented as experimental number of cycles to failure, $N_f$, vs. estimated number of cycles to failure, $N_{f,e}$ including dashed lines representing the experimental data CA scatter bands.
The results presented in Figure 7-37 makes it evident that the TCD method achieved a high level of accuracy with fatigue life predictions always falling within the experimental data CA scatter bands. In more detail, the PM, LM, AM and VM were found to give comparable fatigue strength estimates in the medium- and high-cycle fatigue regions with overall maximum error interval of ±19%, ±9%, ±16% and ±12%, respectively. However, it can also be noted that the TCD method predictions were slightly on the conservative side. This could be an evidence that slightly greater average rivet clamping force values than 140 MPa may be more representative of the clamping force magnitude which was present in the rivets of the tapered cross-girders tested by Bassetti et al. [60]. This also agrees well with the results of experimentally measured rivet clamping forces in [60] obtaining pre-tension forces ranging between 140 to 160 MPa in two of the cross-girder rivets.

On the other hand, the S-N method results were found to be sensitive to the choice of the detail classification adopted for the fatigue assessment. Different detail classifications used
in this case study predicted fatigue lives which could be characterised with high degree of conservatism in the case of Modified Class B estimates always falling outside the CA scatter bands. Detail Category 71 and Class D were capable of estimating fatigue lives within the CA scatter bands.

In overall, the level of accuracy shown by the TCD method for fatigue assessment of full-scale riveted girders is promising, since, from the statistical point of view, a predictive method cannot be expected to be more accurate than its associated experimental data. The findings of the case studies investigated in this and the previous chapter have aided to provide the confidence in reliability and the applicability of the TCD method in fatigue assessment of simple and more complex riveted details. The accuracy of the TCD method in estimating fatigue life was found not to depend on the geometry and complexity of the investigated detail or loading condition despite the fact that a general $L$ vs. $N_l$ relationship calibrated for a structural wrought-iron material was adopted in fatigue assessment process. Therefore, considering the results of the investigated riveted details, it can be concluded that the TCD method can be safely applied to assess fatigue performance of riveted bridge details subjected to simple or complex fatigue loading. In the next chapter, the focus is on applying the TCD method in fatigue analysis of one of the most fatigue-prone details commonly encountered in riveted railway bridges. Fatigue life of a stringer-to-floor-beam connection is investigated in the next chapter, through a global-local FE model, to assess the accuracy of the TCD method as well as the S-N approach compared to the corresponding experimental data obtained.
7.7 Concluding remarks

In this chapter, the validity and applicability of the TCD method in fatigue assessment of old metallic riveted built-up bridge girders subjected to CA fatigue loading were investigated. The TCD method applied in the form of the PM, LM, AM and VM was used for fatigue life estimation of such specimens. Additionally, the traditional S-N method as a more commonly used fatigue assessment method proposed in various codes of practice was also used to estimate fatigue life of investigated specimens to create a comparison basis.

The novel formalisations of the TCD method were seen to be extremely successful in predicting the fatigue life of full-scale riveted built-up girders subjected to uniaxial cyclic bending stresses. The level of accuracy observed in the TCD predictions for full-scale girders was higher than those seen when the TCD was applied for fatigue assessment of flat plates with a circular hole(s) (presented in the previous chapter).

Also, the TCD predictions were seen to have higher accuracy and consistency as compared to the estimates of the S-N approach. The results of the TCD method in the medium- and high-cycle fatigue regime were seen to always fall inside the experimental data CA scatter band compared to the S-N approach predictions always falling on the conservative side and in many instances outside the CA scatter band. The S-N method predictions were found to be highly sensitive to the choice of the detail classification used for fatigue assessment. For instance, Class D and Detail category 71 were the only detail classifications capable of predicting fatigue lives, despite having some degree of error, but in most of the cases inside the scatter bands. This confirms the higher predictive capability of the TCD method in fatigue assessment of such riveted components compared to the more commonly applied S-N approach. In the cases where the S-N method gave the least conservative fatigue life predictions, the results of Class D, Detail Category 71, WI-rivet Class and Modified Class B in the medium- and high-cycle region were found to be conservative up to a factor of about 2, 3, 5 and 6.5, respectively, as compared to the results of the VM formalisation.

These results seem particularly promising since in the majority of the presented case studies no material-specific plain S-N curve of the girder material was available to utilise in fatigue assessment to the TCD method. In such cases, the use of plain material S-N data obtained from the literature for the same material type, generated at the same stress ratio as the one damaging the investigated material, for fatigue predictions to the TCD method was shown
not to adversely affect the accuracy of the estimates. However, it was also noted that the predictions are slightly sensitive to the choice of the selected plain S-N curve used for fatigue life assessment (for instance, see Figures 7-6 (a) and (d)).

Overall, it was confirmed that adopting the $L$ vs. $N_t$ relationship calibrated for the wrought-iron material of the Chepstow Bridge (see Figure 3-4) can still lead to highly accurate predictions when the TCD method is used for fatigue assessment of wrought-iron or mild steel riveted built-up girders. This is important as in many realistic cases, for an old riveted bridge which is still in service, performing adequate fatigue tests on plain and notched material obtained from that bridge to establish material-specific $L$ vs. $N_t$ relationship is not always possible.

Another conclusion worth mentioning is that knowing the S-N behaviour of the full-scale riveted built-up girder, the TCD method was seen capable of estimating the rivet clamping force in the rivets of the girders. This was verified by comparing the rivet clamping force estimated by the TCD against the rivet clamping force experimentally measured for the Vindelälven Bridge built-up stringer [46] as well as the Hinterrhein Bridge tapered built-up girders [60].
8 Fatigue life evaluation of riveted stringer-to-floor-beam connections

8.1 Introduction

As mentioned in the previous chapters, fatigue performance of old riveted metallic bridges (because of their considerable age, increase of demand for heavier freight trains and traffic volume) has been the subject of interest in the past few decades. Majority of the available fatigue related studies are concerned with primary members of such old structures (for instance, cross-girders or stringers) owing to the crucial load-carrying role played by these elements. However, experience and research have shown that connections between different members of these bridges are often more prone to fatigue damage [7, 10]. A frequently reported example of such fatigue-prone details is the stringer-to-floor-beam connection in riveted railway bridges (see Figure 8-1 and Figure 8-2). These connections are typically constructed using double angles which are coupled to the web of the stringers as well as the floor beam by the means of rivet fasteners. These types of connections are primarily used to transfer the live load-induced stringer end reaction to the floor beams. Hence, they are designed with respect to the corresponding shear force and idealised as pin connections.

Figure 8-1 Typical floor system connections used in railway bridges [82]
However, in reality, since a certain degree of end rotational stiffness exists in the double angle connections, there are two mechanisms that can result in the development of fatigue damage caused by secondary deformation-induced flexural stresses in these connection types.

*Figure 8-2 A typical stringer-to-floor-beam riveted connection in railway bridges [82]*

In the first mechanism, shown in Figure 8-4, the consequent stringer-end moment (generated by vertical bending of the stringer under primary load action) causes out-of-plane distortion of the outstanding legs of the connection angle over the entire depth, with the maximum deflection occurring at the top, and decreases as the distance from the top of the angle increases. Under this mechanism, the rivets of the connection angle outstanding legs can also be subjected to axial tension. The amount of this axial tension declines as the distance from the top of the angle increases [63].

The second mechanism is caused by the deflection of the bridge main girders under the passage of a train, resulting in the transverse horizontal deformation of floor beams, which in turn introduces lateral flexural and axial stresses in the outstanding legs or the rivets of the connection angles.
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These mechanisms are likely to generate fatigue cracking in the outstanding legs (typically at the fillet) and the rivets connecting the stringer to the floor-beam web in the upper end of the double-angle connection [12].

![Distortion-induced secondary bending stresses in the outstanding leg of the connection angle caused by the stringer-end rotation (ϕ) [67]](image)

Fatigue damage at the bottom of the stringer-to-floor-beam connection in riveted highway and railway bridges is also reported in the literature [12]. This fatigue damage, as shown in Figure 8-4, occurs if the floor beams possess high flexural flexibility. Consequently, live load-induced vertical deflection of the floor-beams (bending deflection) results in a positive bending moment in the stringer-to-floor-beam connection angle with the upper part of the connection being in compression and the lower parts being subjected to deformation-induced flexural stresses. The existence and magnitude of these stresses mainly depend on the flexural flexibility of the floor beams. In this mechanism, fatigue cracking occurs in either the outstanding legs and/or the rivets in the lower end of the connection angle [12].
It should be noted that the ultimate load-carrying capacity of the double-angle connection would not be adversely affected if these connections were designed as such to allow adequate rotational flexibility to avoid generation of stringer-end moments. However, as mentioned above, the stringer-end rotational stiffness creates high local stresses in different parts of the connection angle which can potentially impair the fatigue performance of such connections if repeated many times [63].

In the preceding chapters, the validity, applicability and accuracy of the Theory of Critical Distances (TCD) were investigated when applied to assess the fatigue life of simple and complex wrought-iron and mild steel riveted bridge details. In cases where experimental fatigue test data were available and were possible to estimate the S-N behaviour of the investigated details under constant amplitude loading, the TCD was shown to be capable of making more accurate fatigue life predictions within constant amplitude (CA) scatter bands as compared with the traditional S-N approach. In all the case studies, the material characteristic length versus number of cycles to failure relationship (L vs. Nf) required according to the TCD method was established using the calibration procedure explained in Figure 3-2. The above L vs. Nf relationship (see Figure 3-4) corresponds to the experimental S-N data collected from [38] for the structural wrought iron material of Chepstow Bridge. Despite the lack of sufficient S-N data on the plain and notched specimens for case specific calibration of an L vs. Nf relationship, the TCD method results were seen to be within the

Figure 8-4 Stringer-end rotation at the lower end of the connection caused by flexible floor-beam [83]

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target error factor of ±20% and within the CA scatter bands determined for the parent material. This confirms the predictive capabilities of the TCD method with regard to old structural metal fatigue estimation.

In this chapter, initially, the CA fatigue tests carried out by Al-Emrani on mild steel riveted stringer-to-floor-beam connections of the Vindelälven Railway Bridge are presented [7, 55, 63]. An S-N behaviour could not be estimated for the double angle connection of the Vindelälven Bridge because the number of fatigue test result is limited and a single stress range level is used in all the tests. However, some pieces of information regarding the fatigue crack initiation and propagation in the connection angles (near the fillets at the level of top rivets) as well as fatigue cracking and failure of rivets are available. Therefore, it becomes interesting to determine whether the linear-elastic TCD method is capable of estimating the fatigue-critical locations (fracture initiation positions) and accurate predictions of the number of cycles to fatigue failure for the critical members. For performing fatigue life estimation based on the TCD method, a 3-D finite element (FE) model of the Vindelälven Bridge double angle connection is created. The behaviour of the FE model of the connection (e.g. stiffness) is then verified against the results of the experimental static tests carried out on the actual specimens in [7, 63]. Eventually, the TCD methods in the form of the PM, LM, AM and VM are used to predict the fatigue life of double angle connections at various critical locations. Moreover, the S-N approach as a more frequently used fatigue design and assessment method (suggested in many fatigue design and assessment codes of practice) is also adopted for fatigue assessment of the same connection detail. Fatigue assessment based on the S-N approach is performed using three different S-N curves ‘Class D’ [28], ‘Detail category 71’ [27] and ‘modified Class B’ [28] in conjunction with nominal stress range obtained from the FE analysis of the stringer-to-floor-beam connection model.

The predicted fatigue lives of the S-N approach and the TCD method are presented as S-N curves for different hot-spot locations. The predicted fatigue lives are also compared with the available corresponding experimental data to quantify the differences in the estimates and measure the accuracy levels of the above-mentioned fatigue assessment methods.
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8.2 Experimental fatigue tests on Vindelälven Bridge stringer-to-floor-beam connections

In the Literature Review chapter, a summary of constant amplitude fatigue tests by Al-Emrani on eight built-up stringers of the Vindelälven riveted Railway Bridge was presented [7, 55]. It was mentioned that one of the main objectives of the experiments was to determine the efficiency of drilling a stop hole as a temporary remedy to arrest fatigue crack propagation. This chapter, however, is focused on investigating the fatigue performance of the double-angle riveted ‘shear’ connections joining these built-up stringers to the cross-beams of the Vindelälven Bridge. The riveted railway bridge over the Vindelälven River was constructed in 1896 at Vännäsby in Sweden. The bridge consisted of three simply supported arch-shaped truss spans (71.2 meters each), which were replaced in 1993 because of insufficient load-carrying capacity. The results of tensile tests on flange material taken from the riveted girders exhibited a mild steel material with an average yield stress, $\sigma_y$, equal to 278 MPa, an ultimate tensile stress, $\sigma_{UTS}$, equal to 425 MPa, and a 35.2% elongation at fracture [48].

The results of an extensive full-scale experimental study by Al-Emrani on fatigue performance of riveted stringer-to-floor-beam connections of the Vindelälven Bridge are published in [7, 63]. The objective of the study was to assess the effects of secondary-induced stresses on fatigue crack initiation and propagation in such ‘shear’ connections.

Three full-scale bridge ‘parts’ (specimens I, II and III) which were taken from the same span of the old Vindelälven Bridge were used in the experiment. Each test specimen consisted of four longitudinal stringers attached to three transverse floor beams by means of L-profile riveted double-angle connections. The test specimens also had the sway- and cross-bracing elements of the original bridge attached to it as well as the remains of the tension chord wind-bracings.

The details of the riveted double-angle connection linking the stringers to the floor-beams of the Vindelälven railway bridge are shown in Figure 8-5. Bearing was positioned under each end of the test specimen floor beams at the location where the stringers meet the floor-beam. The specimen was loaded using four hydraulic jacks such that each stringer was subjected to a four-point bending with the actual point load being applied at each stringer’s centre line and delivered to it via a distribution beam. The loading condition was assumed to simulate the effect of a train passage.
The sketch in Figure 8-6 demonstrates the test set up and loading condition used in the static and CA fatigue tests performed by Al-Emrani at Chalmers University of Technology [67]. All three test specimens were in relatively good condition with only slight corrosion spotted near the connection between sway- and cross-bracing elements and the stringers. The stringer-to-floor-beam connections appeared to be assembled on site using a field-driven riveting process to attach the connection angle to the floor-beam web. A close examination of rivet holes revealed that punching method had been used to form the holes in the connection angles [12]. Paint layer on the connection angles of specimens II and III were removed to check for any service life fatigue damage before the testing started. Pre-existing cracks of 10 mm and 20 mm in length were found near the fillet of the connection angles at the level of the upper rivet for specimens II and III, respectively. For specimen I, the paint layer over the critical fillet section of the connection angle was removed after about $2 \times 10^5$ loading cycles and revealed several fatigue cracks with some as long as 60 mm, which indicates connections angles were highly strained during the service life of the bridge [12].
All three experimental tests were performed at the same stress range of +0.44 with load range of $P = 100$ kN being applied to all three specimens. The corresponding maximum and minimum applied loads were 180 kN and 80 kN, respectively. Both before and in between the fatigue testing static tests were also carried out to allow observation of fatigue damage effects on the behaviour of the stringer-to-floor-beam connections. The results of static tests completed prior to fatigue testing were used to evaluate the rotational stiffness as well as the moment capacity of these double angle connections. The graph in Figure 8-7 reprinted from [67] demonstrates the negative bending moment ($M_P$) developed over the middle support in the three tested specimens as a result of corresponding applied load. The value of $M_P$ was estimated using the measured tensile stresses at the extreme fibre at the mid-span of each stringer. The specimens behave as partially continuous which could lead to out-of-plane distortion-induced secondary flexural stresses in such connections. The following expressions were used in [67] to determine the degree of continuity in the tested double-angle connections:

Figure 8-6 (a) Test set up and overall dimensions of each test specimen. (b) Detail of the angles in the double-angle connection [84]
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\[ \alpha = \frac{M_P}{M_f} = \frac{1}{1 + R} \quad \text{Eq. 8-1} \]

where

\[ R = \frac{(3EI/L)_{\text{stringer}}}{K_{\text{rot}}} \quad \text{Eq. 8-2} \]

where \( M_P \) is the negative bending moment over the middle support of a partially continuous two-span girder, \( M_f \) is the negative bending moment over the middle support of a fully continuous two-span girder, \( K_{\text{rot}} \) presents the rotational stiffness of each double-angle connection, \( E \) is Young’s modulus of the stringer material, and \( I \) is the second moment of inertia of the stringer cross-section.

Figure 8-7 Moment-load curves for the three experimentally tested specimens obtained using measured stringer mid-span bending stresses [67]

The size of the \( M_P \) developed in these stringer-to-floor-beam ‘shear’ connections is noteworthy. For instance, for an applied load of \( P = 100 \text{ kN} \), the tested specimens were seen
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to develop bending moment values between 56% and 67% of a fully continuous two-span girder. These correspond to rotational stiffness values, $K_{rot}$, of between $2.2 \times 10^5$ kNm/Rad and $3.5 \times 10^5$ kNm/Rad, respectively. The slight nonlinear softening and hardening trends in the moment-load curves for specimens I, II and III were related to possible local plastic deformation occurring at the vicinity of stress concentration points (e.g. angle fillets). Also, presence of clamping forces in the rivets, pre-existing fatigue cracks in the outstanding leg of the connection angles and mechanical imperfections in between different components of the connections (such as, misalignment, misfit, and initial gaps) were also suggested to have contributed to the nonlinear moment-load behaviour observed in Figure 8-7, particularly that this behaviour seems to have started at relatively low loading levels. Al-Emrani [67] concluded that, minor mechanical imperfections that caused little to no change to the ultimate load-carrying capacity of such connections, might, however, have a substantial impact on their behaviour at lower loading stages.

The cyclic loading applied on the tested specimens caused stringer-end rotation which was restrained by the flexural stiffness of the outstanding leg of the double-angle connection. This was accompanied by an out-of-plane distortion of the outstanding leg with maximum deformation occurring at the top of the angle and reducing towards the bottom of the angle leg. Therefore, the pre-existing fatigue cracks (at the level of top rivet) and previously reported failed rivets (mainly top rivets) in these specimens could be justified by the flexural bending and axial stresses caused by this out-of-plane deformation. The magnitude of out-of-plane deformation at the top of specimen I was only 0.2 mm for an applied load of $P = 180$ kN. This is while the corresponding measured local flexural stresses in the outstanding leg at the level of the top rivet in the vicinity of the angle fillet were the highest and reached a considerable value of about 140 MPa.

The propagation of pre-existing and new fatigue cracks in the connections was monitored in [67]. As Figure 8-8 shows, in specimen I, a fatigue crack of about 50 mm in length was detected near the fillet of the outstanding leg at the level of top rivet after only $2 \times 10^5$ loading cycles. The propagation stage of this crack to a length of about 100 mm was rather rapid and was recorded at less than $5 \times 10^5$ applied cycles. At this stage, the crack growth rate was slightly reduced in a way that development of the same crack to a length of around 140 mm took another $10^6$ loading cycles. The reduction in propagation rate was associated with a local reduction in the flexural stiffness at the top rivet level because of the presence of a developing
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fatigue crack. Eventually, this crack was self-arrested at its full length of roughly 150 mm attained at less than $2.5 \times 10^6$ total applied load cycles. The reduction of flexural stiffness at the top rivet level was followed by an increase in local bending stresses at the next stiffest location in the outstanding leg of the connection angle (second top rivet level). Its result was the initiation of a fatigue crack at this location near the fillet which was spotted at a length of 25 mm after around $1.3 \times 10^6$ loading cycles (see (2) in Figure 8-8). The propagation of this crack in an up- and downward fashion to a self-arrest condition was also rapid in just close to $2 \times 10^6$ cycles (3). At which point, the top and bottom fatigue cracks joined and formed a roughly 300 mm long vertical crack.

![Figure 8-8](image)

*Figure 8-8 Typical propagation scenarios for fatigue cracks near the fillet of the outstanding leg of connection angles [67]*

It was concluded that the reduction in the initial rapid fatigue crack propagation rates was caused by the decline in connection rotational stiffness. These simultaneous changes occurred as a result of a gradual increase in the length of the same fatigue cracks, which consequently reduced the degree of continuity of the double-angle connection to ‘$\alpha$’ values equal to 0.43 (corresponding to a rotational stiffness of about $1.2 \times 10^5$ kNm/Rad) [67].
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other words, gradual reduction in rotational stiffness of the connection angle led to lower bending moment values developing over the middle support of the central cross girder as well as a pronounced gradual increase in the bending moment stresses at the mid-span of the stringers. This trend continued until the fatigue cracks at the level of top two rivet rows were fully arrested which occurred at less than $4 \times 10^6$ cycles in case of specimen I (see Figure 8-8).

A similar fatigue damage development scenario at the angle fillet was observed in the double-angle connections of other tested specimens [67].

The second mode of fatigue failure was observed in the rivets connecting the outstanding legs of the connection angles to the floor-beam web [67]. These rivets were originally designed to withstand only shear forces while, in reality, were also subjected to additional axial and bending stresses because of the out-of-plane distortion of outstanding leg of the connection angle (see Figure 8-9). These bending stresses were amplified at the junction between the rivet shank and its head (point of sharp change in geometry and high stress concentration) and could cause fatigue crack initiation and eventually detachment of the rivet head. This type of rivet failure was noticed in all three specimens tested in [67].

In the case of specimen I, the fatigue failure in the top rivet was developed at $0.9 \times 10^6$ cycles while a relatively low nominal stress range in the magnitude of $\sigma_r = 45 \text{ MPa}$ in its rivet shank was measured. Similarly, fatigue-triggered rivet head fracture was observed at much earlier points for specimen III in the two rivets of the top row at 0.15 and 0.5 million cycles respectively. Pre-existing fatigue damage was detected in these failed rivets after a microscopic examination of their fracture surfaces, which proved the presence of old fatigue cracks prior to commencement of fatigue loading [67]. The loss of the rivet head in the top rivets was seen to induce a reduction in the rotational stiffness of the connections and consequently increase stringer mid-span bending stresses. This effect was analogous to what was experienced because of crack propagation near the fillets of connection angles.

For specimen II, fatigue failure in rivets connecting the outstanding leg of the connection angle to the floor-beam web demonstrated a somewhat diverged behaviour. The head of the rivets on one side of the connection was detached from its shank in a successive manner starting at the top row and moving downwards. In total eight of the ten rivets suffered such fatigue failure by the end of the fatigue test. Other than this noticeable difference, the behaviour of the connection in terms of measured stresses at the stringer mid-span (in other
words, moment-load relationship over the central floor-beam and degree of continuity, \( \alpha \) did not differ considerably to that of the other two tested specimens [12].

In general, very low fatigue life of between \( 0.15 \times 10^6 \) and \( 0.9 \times 10^6 \) cycles was observed for rivets fastening the outstanding leg of the connection to the floor-beam web in the stringer-to-floor-beam connection of the Vindelälven Bridge. This was suggested to be partly because of some pre-testing accumulated old fatigue damage detected in the microscopic examination of the fracture surface and partly because of the fairly low rivet clamping force values in these rivets. In more details, strain measurement performed on two different rivets located at the top row of the tested connection angles revealed a clamping stress of about ‘30 MPa’ to be present in these rivets [7, 63].

Based on the observation of results for static and fatigue tests on three full-scale stringer-to-floor-beam connections, it was concluded that, high rivet clamping force values did not contribute greatly in reducing the connection stiffness or flexural stresses in the outstanding leg of such connections. On the other hand, in this type of fasteners, higher values of rivet clamping force could result in extensive decrease in the axial and bending stresses that might exist in the rivets because of the out-of-plane distortion of the angles [12].

In summary, two fatigue failure mechanisms were detected in the tested double angle connections, both of which were associated with the out-of-plane deformation of the outstanding leg of the connection angle (see Figure 8-3). The first mode of failure was produced by the flexural bending stresses and leads to fatigue cracking of the outstanding leg near the fillet at the level of the top rivet rows. The second failure mechanism was caused by the secondary bending and axial stresses formed as a result of prying action and bending of the rivets. This latter mechanism leads to fatigue crack initiation and eventually full rivet head detachment in the critical rivets connecting the outstanding leg of the angle to floor-beam web.

### 8.3 Finite element (FE) analysis of Vindelälven Bridge stringer-to-floor-beam connection

In [84], the results of a finite element analysis on the stringer-to-floor-beam connections of the Vindelälven Bridge was published. This FE analysis was aimed to investigate the performance of the tested connections and to gain a more in-depth understanding of their behaviour. Based on the outcome of the FE analysis, it was confirmed that these double-angle
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‘shear’ connections possessed fairly high stringer-end rotational stiffness and behaved as partially fixed girders capable of generating a substantial negative bending moment over the middle support. This rotational stiffness of the connection was found to be a consequence of lack of adequate flexural flexibility in the angle legs. The angle leg flexibility itself was seen to be primarily affected by the gauge distance (defined as the distance between the rivet centre line in the outstanding leg of the angle to the back of the angle fillet). However, it was noted that parts of the angle at the level of rivets contribute the most to the connection stiffness and causes a non-uniform distribution of flexural bending stresses generated because of secondary out-of-plane distortion.

In the FE analysis of the stringer-to-floor-beam connections by Al-Emrani and Kliger [84] three different clamping stress values of 30 MPa, 65 MPa and 140 MPa were used to investigate its effect on connection behaviour. The increase of the rivet clamping force was shown to have very little effect on reducing the rotational stiffness of the connection or the flexural stresses at the angle fillet. However, higher rivet clamping pre-tension values were found to considerably reduce the ‘stress range’ in the rivet caused by the external applied loads. Finally, it was verified in the FE analysis [84] that the clamping forces present in the rivets connecting the outstanding leg of the angle to the floor-beam web in the tested specimens were in the order of magnitude of about 30 MPa.

Similar FE analysis was also performed in this study on the stringer-to-floor-beam connections of the Vindelälven Bridge. The primary objective in this FE analysis was to study the fatigue performance of these connections through the TCD and the S-N methods. The data obtained from the experimental data with regard to the connection stiffness were used to verify the results of the FE model.

Similar to [84], three rivet clamping stress values of 30 MPa, 65 MPa and 140 MPa were also used in the FE model in this study. This was mainly to evaluate the effect of increasing the rivet clamping force on fatigue life in such connection.

8.3.1 Description of the FE model

The shell-to-solid coupling technique in the commercial FE-package Abaqus/CAE v6.10 [39] was used to create the 3-D FE model of the stringer-to-floor-beam connection. The FE model adopted in this investigation is shown in Figure 8-9. To be more economical on the
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computational cost of the analysis, symmetry planes were assumed, and the model was reduced to a quarter of the actual size. The first plane of symmetry on the Y-Z axis passes through the centreline of the stringer while a second one exists on the X-Y axis through the centreline of the floor-beam web.

![FE model](image)

**Figure 8-9** The FE model used for the analysis of the Vindelälven Bridge stringer-to-floor-beam connection

The material properties of the mild steel used in the stringers were determined through a series of tensile tests on flange material in a previous study [48]. The test results displayed an average yield stress, $\sigma_y$, of 278 MPa, an ultimate tensile stress, $\sigma_{UTS}$, of 425 MPa, and a 35.2% elongation. The mild steel was assumed to have linear-elastic behaviour during the analysis with Young’s modulus and Poisson’s ratio values of 200 GPa and 0.3, respectively.

Because of the lack of available data on the size of some of the connection components, assumptions and simplifications had to be adopted in the modelling process. For instance, the rivet head dimension and the angle fillet radius were not stated in the original published study [12]; therefore, some presumed values based on similar cases found in the literature had to be used. The rivet head radius was assumed to be 7 mm larger than the shank radius. This assumption was in good agreement with a similar value used in the FE analysis of a stringer-
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to-floor-beam connection by Imam [11] and other available sources [85]. The fillet was included in the FE model with a radius of 6 mm based on typical field measurements [86].

![Close-up image of the built-up double angle connection adopted in the FE analysis](image)

Figure 8-10 Close-up image of the built-up double angle connection adopted in the FE analysis

More details on the dimensions used when modelling the stringer-to-floor-beam connection is presented in Figure 8-5 and Figure 8-6. The gauge distance, ‘g’, defined as the distance from the backside of the angle leg to the centreline of the rivet hole in the outstanding leg, was taken equal to 50 mm. The vertical spacing between the centreline of the adjacent rivet holes in the outstanding leg of the angle leg was modelled equal to about 140 mm. The vertical distance from the centreline of the edge rivet hole to the angle top/bottom edge was taken equal to 90 mm.

The ideal zero clearance was assumed for the rivet shank and the associated rivet hole. In other words, the lateral expansion of the shank material during the hot-driven rivet forging process filled up the gap between the shank and rivet hole. The possible misfit, misalignment
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of rivets/rivet holes, rivet size variations and other mechanical imperfections that might have existed in the tested specimens was not considered in the FE model.

Figure 8-11 Close-up image of the FE mesh used to model the solid stringer-to-floor-beam connection

Various components of the stringer-to-floor-beam connection in the FE model are shown in Figure 8-10. In the generated FE model, with 300 mm of the built-up stringer end consisting of one top and bottom angle, the web plate and the corresponding rivets were also included. As seen in Figure 8-6 and Figure 8-8, the outstanding leg of the angle contains a chamfer edge at the top and bottom. Since the precise vertical and horizontal dimensions of these chamfer edges were not known, suitable dimensions in the modelling were assumed as such to allow for a structured mesh type at the level of the top rivet row. This is particularly of great importance since the angle fillet was reported as one of the most fatigue-prone locations in such connections [12].
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Also, looking at Figure 8-6 and Figure 8-8, the L-shape angles connecting the stringer and floor-beam webs in the tested specimens appear to be slightly bent close to the upper end to allow the backside of the angle to sit flat on the stringer web. This was because the top portion of the angles overlap the top flange of the built-up stringer. However, for simplicity in the FE modelling, this feature was not considered. Instead, as shown in Figure 8-10, an 11 mm spacer plate was used to fill the gap between the stringer web and the back side of the angle leg. The consequences of these simplifications and assumptions on the FE analysis results are investigated in the FE model verification section.

The main portion of the stringer, apart from the 300-mm end section, was modelled using 8-noded shell elements (S8R). The shell model was created by transforming the stringer cross-section (shown in Figure 8-6) into an equivalent I-section keeping the second moment of inertia and total depth constant. The above I-section consisted of an equal-size top and bottom flange of 238 mm wide and 5.5 mm thick and a web plate of 819 mm deep and 25.897 mm thick.

A 20-noded solid element with reduced integration (C3D20R) was used to model the angle, rivets and 300 mm of the stringer end. This element type was selected based on the results of the benchmark study performed in earlier stages of this study (see Chapter 5). A close-up of the FE mesh adopted to model the solid stringer-to-floor-beam angle connection is shown in Figure 8-11. A total number of about 71,000 quadratic hexahedral elements of type C3D20R was used in the FE mesh of the angle alone.

Thirty-six elements were utilised at the perimeter of the top two rivet holes at the upper part of the angle. In case of rivets fastening the outstanding leg of the angle to the floor-beam web, 32 number of elements were used around the perimeter. To form a structured mesh around the perimeter of the top two rivet holes (critical hot spots) in the outstanding leg of the angle, six circular partitions at 1 mm distance from each other were created around each (see Figure 8-12). The primary intention was to allow creation of an adequate number of stress readout points when performing fatigue assessment based on the TCD method.
Figure 8-13 shows the mesh refinement adopted to adequately capture the linear-elastic stress field in the vicinity of the angle fillet surface as well as the junction between the rivet shank and its head. These locations were reported as most fatigue-damage critical in the experimental tests performed on the stringer-to-floor-beam specimens [67]. The degree of mesh refinement near the critical region becomes increasingly substantial specially when the TCD method is adopted to determine the high-cycle fatigue life. The characteristic length value for old structural metal in the high-cycle region was determined equal to be 1.7 mm (see Figure 3-4). When using the AM or the VM to calculate the amplitude of the reference stress ($\Delta \sigma_{\text{ave}}$) to predict fatigue life, it is reasonable to ensure more than one element falls within the critical area/volume to achieve higher accuracy.
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Figure 8-13 Degree of mesh refinement used at (a) the angle fillet location and (b) the critical rivets

To prevent one model surface from penetrating another, the standard Surface-to-Surface contact function in Abaqus 6.10 [39] was used to define contact between contacting surfaces using finite sliding and master-slave surface algorithm. A total of 61 contact pair surfaces were defined between contacting bodies (rivet-to-connection angle, rivet-to-spacer plate, rivet-to-stringer web, rivet-to-floor-beam web, connection angle-to-floor-beam web, connection angle-to-spacer plate, connection angle-to-stringer top angle, spacer plate-to-stringer web, spacer plate-to-floor-beam web and stringer web-to-floor beam) using a Coulomb-friction formulation with a coefficient of friction, \( \mu \), equals to 0.3 and normal contact properties defined as ‘Hard Contact’ that enables surface separation after contact.
To investigate the effect of rivet clamping force on connection behaviour and also fatigue life estimates, three different rivet clamping force values of 30 MPa, 65 MPa and 140 MPa were used in the FE analysis. In the FE model, the clamping force was defined at mid-length of the rivet shank using the ‘PRE-TENSION SURFACE’ option (BOLT LOAD) in Abaqus, which applies a concentrated force over a user-defined cross-section surface.

The shell-to-solid coupling feature as instructed in Abaqus 6.10 [39] was used to couple the shell-to-solid interface. Abaqus creates constrains that couple the displacement and rotation of the shell node to the average displacement and rotation of the solid surface in the vicinity of the shell node.

As to the boundary condition, the base of the floor-beam web was constrained in the X-, Y- and Z-directions to form a pin support and also to prevent free-body rotation of the whole model.

Faces of the stringer web (both solid and shell), rivet ends and floor-beam web which coincide with the Y-Z symmetry plane were assumed to have null displacement in the direction normal to this plane and hence their translation was restricted in X-direction (see Figure 8-9). Similarly, the faces of the floor-beam web and rivet ends coinciding with the X-Y symmetry plane were assumed to have zero displacement in the direction normal to this plane and hence zero deformation in X-direction. Finally, the bottom flange of the shell stringer model was constrained against translation in the X- and Y-directions at the opposite end to the connection angle. This boundary condition was presumed to closely reproduce the experimental condition adopted in the stringer-to-floor-beam connection tests conducted by Al-Emrani [12].

A total of 3 FE models were created with about 10 general static steps in each model. The models were identical in every aspect with the only main difference being the magnitude of the rivet pre-tension force applied to the rivet shanks. The boundary condition and the contact interactions were defined in the initial step and were allowed to propagate throughout the analysis. In the first general static step of the analysis, only the clamping force was applied. The initial time increment size in step 1 was set to $5 \times 10^{-8}$. Automatic adjustment for subsequent time increments during this step was enabled to alter the size of the remaining increments based on how quickly the solution converged.
Beginning with step 2 in the analysis, the shell stringer was subjected to a point load of $P/2$ at two locations specified in Figure 8-6. The applied load, ‘$P$’, values of 30, 40, 50, 80, 100, 120, 150, 180 and 200 kN were used in subsequent steps, respectively. The effect of the load applied in each step was not permitted to propagate to the succeeding step. This allowed for a separate load case scenario to be investigated in each analysis step.

### 8.3.2 Verification of the FE model

The illustration in Figure 8-14 displays the calculated and measured nominal stresses obtained from the FE analysis and ‘specimen I’ respectively for an applied load of $P = 100$ kN.

*Figure 8-14  Calculated versus measured nominal stresses along the stringer depth at a 150 mm distance from the floor-beam centreline [84]. The calculated stresses are obtained from the FE model with a 30 MPa clamping stress.*
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The neutral axis for the FE analysis results appears to be at a slightly higher position compared to that of the experimental data. Nevertheless, a good correlation exists between the stresses calculated through the FE analysis compared to the one observed in the tested ‘specimen I’ with only about 8% difference in the magnitude of peak stresses.

Figure 8-15 Here, (a) and (b) correspond to the FE model deformed shape and displacement along the A-B line, respectively, for a connection angle under applied load, \( P = 180 \, \text{kN} \), and a clamping force of 30 MPa.

According to the measured displacement in the experimental study on the stringer-to-floor-beam connections [12], the magnitude of the out-of-plane deformation at the top of the ‘specimen I’ was only around 0.2 mm for an applied load of \( P = 180 \, \text{kN} \). This is comparable with the corresponding value calculated near the top of the outstanding leg of the connection angle (\( \delta_{Z,1} \)) in the FE analysis (see Figure 8-15 (b)). This further confirms that the flexural and
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Rotational stiffness demonstrated by the outstanding leg of the angle in the FE model is in close agreement with those observed in the actual tested specimens [12]. The graph in Figure 8-16 also confirms that the rotational stiffness exhibited by the FE model is in line with the measurements recorded in the tests [84]. It also shows that the FE model with a 30 MPa rivet clamping force experienced out-of-plane deformations in the order of magnitudes with those observed in the experiments [12]. The results in Figure 8-16 also show that an increase in the rivet clamping forces from 30 to 140 MPa causes an average reduction of 15% in the deformation induced by the external applied load at the angle top.

![Graph showing measured and calculated displacement at the top of the connection angle](image)

*Figure 8-16 Measured and calculated displacement at the top of the connection angle [84]*

The negative moment \( (M_P) \) developed by the connection FE models are shown in Figure 8-17 as a function of the external applied load, \( P \). These moment-load relationships are obtained for models with rivet clamping forces of 30, 65 and 140 MPa. The value of \( M_P \) was estimated using the bending tensile stresses at the extreme fibre of the stringer in the bottom flange at the points of load application. Similar to the results of the experimental tests on specimens I,
Fatigue life evaluation of riveted stringer-to-floor-beam connections

II and III, slight nonlinear softening behaviour is also observed in the moment-load relationships calculated for the present FE models especially at higher values of applied load. Such behaviour could be associated with possible local plastic deformation occurring at the vicinity of stress concentration points (e.g., angle fillets).

![Figure 8-17](image)

*Figure 8-17  Moment-load relationships for the FE models derived using the bending stresses at the extreme fibre of the stringer tension flange at the location of load applications [67]*

The degree of continuity, \( \alpha \), and the rotational stiffness of the FE models were calculated using the expressions of Eq. 8-1 and 8-2. The rotational stiffness, degree of continuity and negative bending moment values developed in the double angle connection corresponding to an applied load range of \( \Delta P = 100 \text{ kN} \) are presented in Table 8-1 for FE models with 30, 65 and 140 MPa rivet clamping force. It can be seen that all three models behave as partially continuous, and the magnitude of \( M_P \) is comparable to those measured experimentally for specimen I and II investigated by Al-Emrani [12].
Table 8-1 Connection stiffness and negative bending values calculated in the three FE models

<table>
<thead>
<tr>
<th>Rivet Clamping stress $\sigma_{\text{Clamp}}$ (MPa)</th>
<th>Rotational stiffness $K_{\text{rot}}$ (kNm/Rad)</th>
<th>Degree of continuity $\alpha$ (%)</th>
<th>Stringer-end moment $M_P$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>2.5E+05</td>
<td>60.1</td>
<td>61.0</td>
</tr>
<tr>
<td>65</td>
<td>2.7E+05</td>
<td>62.5</td>
<td>63.3</td>
</tr>
<tr>
<td>140</td>
<td>3.1E+05</td>
<td>65.2</td>
<td>66.1</td>
</tr>
</tbody>
</table>

The degree of continuity and rotational stiffness displayed by the FE models fall within the range of stiffness values observed for tested specimens I, II and III [67]. Furthermore, the calculated $K_{\text{rot}}$ values shown in Table 8-1, irrespective of the rivet clamping force magnitude, seem to fit closer to the upper-range values found in the literature for rotational stiffness of such connection types [10, 82].

Looking at the results in Table 8-1, it can also be concluded that, increasing the rivet clamping force from 30 to 140 MPa improved the connection stiffness by about 19%, which corresponds to a 7% growth in the stringer-end moment, $M_P$.

As concluded in the experimental study by Al-Emrani [67], the stiffness of stringer-to-floor-beam connections is primarily governed by the flexural stiffness of the outstanding leg, which in turn is dependent on the gauge length. The connection angle was observed to show higher stiffness at the rivet levels. This phenomenon was also observed in the results of the current FE analyses with the corresponding results for the model with a 30 MPa rivet clamping force subjected to a load range of $\Delta P = 100$ kN shown in Figure 8-18 (a). The flexural bending stress ($\sigma_x$) along the depth of the angle has a nonuniform distribution with peak values reached at the rivet levels.

The distribution of the bending stress across the gauge length and through the angle thickness is shown in Figure 8-18 (b) for both FE models with 30 and 140 MPa clamping forces under the same applied load ($\Delta P = 100$ kN). The out-of-plane displacement of the angle subjects the outstanding leg to flexural stresses which reach a maximum near the angle fillet. An increase in rivet clamping force from 30 MPa to 140 MPa appears to have slightly amplified the stiffness of the connection in the rivet region causing the point of contra-flexure in the angle leg to be shifted towards the fillet. In other words, an increase in rivet clamping stress forms a state of partial fixity at the rivet centreline of the outstanding leg, leading to a behaviour closer to that suggested by the Wilson model [63] (see Figure 4-1).
Figure 8-18 The magnitude and distribution of flexural stresses in the connection angle ($P = 100$ kN) (a) near of the fillet along the angle depth and (b) along the gauge length and through the thickness.
Fatigue life evaluation of riveted stringer-to-floor-beam connections

The results in Figure 8-18 (b) show that an increase in the rivet clamping force from 30 to 140-MPa leads to greater compressive stresses in the angle material under the rivet head as well as higher flexural stresses at the fillet. This could be explained by observations made on the effects of rivet clamping force increase on the deformation change at these two points (marked as ‘A’ and ‘B’ in Figure 8-18 (b)).

For the FE model with a 140 MPa rivet clamping force, the difference between out-of-plane deformation at ‘A’ and ‘B’ in the angle leg was greater compared to that calculated for the FE model with a 30 MPa rivet clamping force. This means that, to accommodate the out-of-plane deformation caused by the applied load of \( P = 100 \text{ kN} \), a greater curvature was formed in the gauge length of the outstanding for the FE model with a 140 MPa rivet clamping force in comparison to that obtained for the FE model with a 30 MPa rivet clamping force. This behaviour explains the higher bending stresses generated at the angle fillet because of the increase in rivet clamping force. This remains true despite the fact that a rivet clamping force increase resulted in a reduction in a deformation (\( \delta_z \)) reduction at the top of the angle with respect to the floor-beam web (see Figure 8-16).

Additionally, the increase in the rivet clamping force is expected to result in a smaller increase in the axial stress developed in the rivets connecting the outstanding leg of the angle to the floor-beam web. In Figure 8-19 the increase in the nominal axial stress in the upper rivet resulting from the external applied load is calculated using the results of the FE analyses and presented here as a function of connection moment. These calculations are made at the centreline of the rivet and at the level of rivet shank to head junction. One clear observation regarding the Figure 8-19 is that, increasing the rivet clamping force from 30 MPa to 140 MPa is seen to reduce the nominal axial stress range in the upper rivet by about 65%. However, the main fatigue failure mechanism for these rivets in the experimental research was noted to be due to a combination of bending and axial stresses caused by an out-of-plane distortion of the angle [12].

A good agreement is found when comparing the FE-calculated results for the model with a 30 MPa clamping force to the corresponding experimentally measured data for ‘specimen I’, also plotted in Figure 8-19 [67]. The results in Figure 8-19 estimates a rivet clamping force of about 30 MPa for the three specimens tested by Al-Emrani [12]. Similar rivet clamping force values
were suggested for the tested specimens by Al-Emrani and Kliger in the FE analysis of stringer-to-floor-beam connections [84]. This further verifies the results of the current FE analyses.

![Figure 8-19 Measured and calculated increase in the nominal axial stress in the top rivet of the outstanding leg of the angle as a function of connection moment [67]](image)

Similar trends were obtained when assessing the nominal axial stress change in the second upper rivet of the connection. Additionally, the out-of-plane deformation (see Figure 8-15 (b)) as well as the flexural bending stresses near the angle fillet (see Figure 8-18 (a)) appear to be considerable, which could explain the fatigue failures developed at these locations during the experimental tests.
Fatigue life evaluation of riveted stringer-to-floor-beam connections

8.4 Fatigue life evaluation of stringer-to-floor-beam connection using the TCD method

This section presents the TCD method for fatigue life estimations for the stringer-to-floor-beam connection with three different rivet clamping forces of 30, 65 and 140 MPa are presented. As for the FE analysis, the required linear-elastic stress field in the global-local model was determined through Abaqus [88]. The FE models were subjected to four-point bending at load levels of, $P$, equal to 30, 40, 50, 80, 100, 120, 150, 180 and 200 kN. The critical distance vs. number of cycles to failure relationship ($L$ vs. $N_f$) determined for wrought-iron material of the Chepstow Bridge [38] (see Figure 3-4) was adopted to estimate the fatigue life of the connection at various hot-spot locations. The results of the experimental investigation performed by Al-Emrani on three stringer-to-floor-beam specimens [7, 63] as well as careful observation of the FE analyses results were used to identify the maximum tensile stress points, that is, hot-spot locations. The experimental research [12] revealed that, the angle fillet sustained very long fatigue cracks initiating at the level of the top two rivets and extending vertically in both directions. Additionally, the rivet shank to head junction, especially in the top two rivets, had pre-existing in-service fatigue cracks and suffered fatigue failure during fatigue testing with their heads popping off. These locations, as depicted in Figure 8-20, Figure 8-21 and Figure 8-22, are investigated in this study as the main hot-spot locations for the purpose of fatigue life estimation. Since loads are applied as static, gravity point loads, the location of the hot-spots is expected to remain the same with no significant change for different load levels.

To use the TCD method, a focus path has to be defined initially. A focus path is a straight line assumed to emanate at the stress concentration surface and extend perpendicular to this surface in a direction that experiences the maximum stress gradient. It is used to obtain the average stress range magnitude, $\Delta \sigma_{ave}$, required when using the TCD method for fatigue life estimation. Here, an alternative method suggested in [89] was adopted to determine the focus path at the location of the mentioned hot-spots. The above method uses the principal of the PM, which ignores the stress at the stress concentration surface and adopts a stress value determined at a distance of $L/2$ away from the surface. As shown in Figure 8-20, Figure 8-21 and Figure 8-22, a curved critical path was drawn at a distance of $L/2$ from the stress concentration surface and parallel to this surface.
Fatigue life evaluation of riveted stringer-to-floor-beam connections

Figure 8-20 Principal stress fields for the angle with a close-up of the critical path located at the high-tensile region of the fillet near the level of the top rivet row.

Figure 8-21 Principal stress fields for the top rivet with a close-up of the critical path located at the high-tensile region.
Fatigue life evaluation of riveted stringer-to-floor-beam connections

Figure 8-22  Principal stress fields for the backside of the angle in contact with the floor-beam web. A close-up of the critical path located at the high-tensile region of the rivet hole at the level of the top rivet row is also shown.

A potential fatigue crack is assumed to occur at a point on this critical path that experiences the largest stress. Hence, the focus path can be defined as a line passing through this high-stress point and perpendicular to the stress concentration surface. To carry out fatigue life prediction using other formalisations of the TCD method, such as, the AM and the VM, the average stresses within a critical semicircular area or hemispherical volume, respectively, placed centrally at the focus path origin are used to determine the $\Delta \sigma_{\text{ave}}$ value. In case of the LM, the stresses on the focus path up to a certain distance ($2L$) away from the stress concentration surface are averaged to determine $\Delta \sigma_{\text{ave}}$.

In this study, all different formalisations of TCD were used to estimate fatigue life for stringer-to-floor-beam connection at six hot-spot locations (i.e., the angle fillet at the level of the top two rivets, the rivet hole edge for the top two rivet holes and the rivet shank to head junction of the top two rivets) by postprocessing the results of the FE analyses. Since plain material S-N data specific to the Vindelälven Bridge mild steel was not available, the BS 5400-10 Class B
Fatigue life evaluation of riveted stringer-to-floor-beam connections

mean S-N curve [28], representative of structural mild steel plain material, was used for the purpose of fatigue life prediction under TCD.

As opposed to the case studies investigated in the earlier chapters, no adequate fatigue life data was available to estimate an S-N curve relationship for the stringer-to-floor-beam connection. This is a common situation encountered in practice where it is required to evaluate the fatigue life of a critical detail without adequate material-specific data (plain and notched S-N data, $L$ vs. $N_t$ relationship). Consequently, it is not possible to present the fatigue life predictions as accuracy diagrams in the form of experimental number of cycles to failure, $N_i$, vs. estimated number of cycles to failure, $N_{i,e}$.

Because of the accuracy of the TCD method, when adopted to estimate the fatigue life of old metallic structural members and connections, was validated and verified in the previous chapters, it is justified in such situations to present the fatigue life predictions in the form of applied load range ($\Delta P$) vs. number of cycles to failure. Therefore, in this investigation, the TCD predicted fatigue life predictions for different components of the stringer-to-floor-beam connection (angle fillet, rivet hole edge and rivet shank to head junction) are presented as $\Delta P$-$N$ curves as shown in Figure 8-24, Figure 8-25 and Figure 8-26. The applied load ranges are the difference between the corresponding minimum and maximum applied loads that the simply supported stringers were subjected to in four-point bending. For instance, for an applied load range of $\Delta P = 100 \text{ kN}$, the corresponding $P_{\text{min}}$ and $P_{\text{max}}$ are 80 kN and 180 kN, respectively.

The results shown in Figure 8-24, Figure 8-25 and Figure 8-26 correspond to the estimated $\Delta P$-$N$ relationship for the stringer-to-floor-beam connection with 30, 65 and 140 MPa rivet clamping force, respectively. To allow for visual comparison, the S-N method’s fatigue life predictions of the stringer-to-floor-beam connection under the same fatigue loading are also plotted in Figure 8-24, Figure 8-25 and Figure 8-26.
8.5 Fatigue life evaluation of stringer-to-floor-beam connection using the S-N approach

This section shows the S-N method for fatigue life estimations for the stringer-to-floor-beam connection with three different rivet clamping force values of 30, 65 and 140 MPa. Fatigue life estimation based on the S-N approach was performed using the S-N classifications presented in Figure 3-1, that is, mean S-N curves of BS 5400-10 ‘Class D’ [28], Eurocodes ‘Detail category 71’ [27], and BS 5400-10 ‘modified Class B’ [28]. One of the main challenges with the S-N method, especially in the case of complex geometries and details such as stringer-to-floor-beam connection, is the determination of a nominal stress value for the critical detail which should be free from stress raiser effects. This stress value is required in conjunction with detail-specific S-N curve for fatigue life assessment according to the S-N method. In this study, the nominal stresses were obtained from the results of the FE analyses of the stringer-to-floor-beam connection models with three different rivet clamping force values of 30, 65 and 140 MPa, each subjected to four-point bending at load levels of, \( P \), equals to 30, 40, 50, 80, 100, 120, 150, 180 and 200 kN.

![Diagram](image)

*Figure 8-23 Position of nominal stress range, \( \sigma_{r,\text{nom}} \), readout points (a) for angle fillet and rivet hole edge hot-spots and (b) for the rivet shank-head junction hot-spot*
Fatigue life evaluation of riveted stringer-to-floor-beam connections

The hot-spot locations shown in Figure 8-20, Figure 8-21 and Figure 8-22 were considered to predict fatigue life based on the S-N method to create a comparison basis against the estimated results of the TCD method for the same hot-spots. For the angle fillet and rivet hole edge at the top and second top rivet row, the nominal stress range ($\sigma_{r,\text{nom}}$) was obtained from the FE results at a horizontal distance of 150 mm from the stringer-to-floor-beam interface at locations (1) and (2), respectively, as shown in Figure 8-23 (a) using red points. The location of the nominal stress range readout point for the rivet shank to head junction hot spot for the top and second top row rivets was at the centreline of the rivet shank and at the level of the rivet shank to head junction as shown in Figure 8-23 (b).

Fatigue life data predicted using the S-N approach are presented in Figure 8-24, Figure 8-25 and Figure 8-26 for connections with 30, 65 and 140 MPa rivet clamping force values, respectively. In the above figures, the S-N method predictions for the angle fillet, rivet hole edge and rivet shank to head junction for both the top and second top rivet row levels are presented in the form of $\Delta P$ vs. predicted fatigue life. The load range values used to present the data are the same as those used for fatigue assessment based on the TCD method. This allows for an easier comparison of the predictions based on the both investigated methodologies.

In Figure 8-24, Figure 8-25 and Figure 8-26, where applicable, the experimental data obtained by Al-Emrani [12] are also plotted (i.e., angle fillet and rivet shank at top rivet level). It should be noted that these experimental data correspond to specimens that had pre-existing fatigue crack accumulation at these hot-spot locations. Despite that, the available experimental data which correspond to $\Delta P = 100$ kN could provide valuable means of comparison between the predictions of the two adopted fatigue life assessment methodologies.
Fatigue life evaluation of riveted stringer-to-floor-beam connections

Figure 8-24  The TCD and the S-N method accuracy in estimating the fatigue life of the connection with a 30 MPa rivet clamping force under four-point bending with $R = 0.44$. Hot spots in top rivet row are shown in (a), (b) and (c), and hot spots in the top second rivet row are shown in (d), (e) and (f).
Figure 8-25 The TCD and the S-N method in estimating the fatigue life of the connection with 65 MPa rivet clamping force under four-point bending with $R = 0.44$. Hot spots in the top rivet row are shown in (g), (h) and (i), and hot spots in the top second rivet row are shown in (k), (l) and (m).
Fatigue life evaluation of riveted stringer-to-floor-beam connections

Figure 8-26  The TCD and the S-N method in estimating the fatigue life of the connection with 140 MPa rivet clamping force under four-point bending with $R = 0.44$. Hot spots in the top rivet row are shown in (n), (p) and (q), and hot spots in the top second rivet row are shown in (s), (t) and (u).
In the following, the predicted fatigue lives at various hot-spot locations in the angle connection are discussed and compared with the available experimental data obtained by Al-Emrani [12]. Figure 8-24 (a) shows the TCD fatigue life predictions at the angle fillet at the level of the top rivet row. The PM, AM and VM results seem relatively similar. Comparison of the fatigue life estimations in Figure 8-24 (a) and (b) reveals that, the TCD predictions at the angle fillet and rivet hole are comparable especially in the case of the VM estimates.

In Figure 8-24 (b), predictions of different formalisations of the TCD at the rivet hole edge appear to have major similarities, which could be because of symmetric distribution of the stress field at this hot-spot location. However, results in Figure 8-24 (a) show higher spread in the predictions with the LM and the VM estimates being the least and the most conservative methods, respectively. This may be the result of the asymmetric stress state observed at the angle fillet on either side of the focus path.

Figure 8-24 (c) presents the TCD estimates at the location of the rivet shank to head junction. The scatter of the predictions is comparable to that of the angle fillet estimates with the PM estimates being 20% and 35% less conservative than those of the AM and the VM, respectively. The predicted fatigue life in Figure 8-24 (c) for the connection model with a 30 MPa rivet clamping force and subjected to an applied load of, $P$, equal to 100 kN, is 0.54, 1.4, 0.42 and $0.33 \times 10^6$ cycles based on the PM, LM, AM and VM, respectively. This conforms well to the results of the experimental investigation [12], for specimens subjected to the same load level, where a fatigue life of between $0.15 \times 10^6$ to $0.9 \times 10^6$ cycles was observed for rivets fastening the outstanding leg of the connection angle to the floor-beam. It should be also mentioned that, pre-existing fatigue damage was detected in the failed rivets in the experimental investigation, which proved the presence of old fatigue cracks prior to the commencement of fatigue loading. This could explain slightly shorter fatigue lives observed in the above experimental data. Despite this, the scatter observed in predictions of different formalisations of the TCD method is seen to fit well in the one obtained in the experimental test results.

At the level of the second rivet row, it is again the rivet shank to head junction that appears to be the most fatigue-critical hot-spot as compared to the angle fillet and rivet hole edge at this level. Figure 8-24 (f) shows the predicted fatigue life for the rivet at the second top row of the connection with a 30 MPa clamping force and subjected to $P = 100$ kN. The estimated fatigue lives are 4.2, 10.2, 3.3 and $2.5 \times 10^6$ cycles based on the PM, LM, AM and VM, respectively. The available experimental results on stringer-to-floor-beam connection [7,63]
does not provide relevant information regarding the fatigue life of the rivet at the second row of the connection to allow for comparison. However, it should be borne in mind that failure of the rivets in the top row alone was observed to release most of the restrained deformation of the connection angle resulting in no further cracking and failure of rivets due to imposed bending [7,63]. Therefore, the fatigue life for the rivets in the second and lower rows would experience improvements if the deformation restraint release took place when the heads of the rivets in the top row popped off.

As mentioned earlier in this chapter, pre-existing fatigue cracks of about 20 to 60 mm long were detected in the vicinity of the angle fillet at the top rivet level of the stringer-to-floor beam connections tested by Al-Emrani [12]. For an applied load of, \( P = 100 \, \text{kN} \), it took about 2 to \( 4 \times 10^6 \) cycles for the pre-existing fatigue cracks in different specimens to grow another 100 mm in length and a further 0.5 to \( 3 \times 10^6 \) cycles to reach a complete arrest. As shown in Figure 8-24 (a), for a connection FE model with a 30 MPa rivet clamping force and subjected to an applied load of, \( P = 100 \, \text{kN} \), the TCD predictions in the form of the PM, LM, AM and VM were 3.9, 12.5, 4.3 and \( 2.5 \times 10^6 \) cycles, respectively. The above TCD predictions agree well with the fatigue crack propagation times in the tested specimens.

Figure 8-25 and Figure 8-26 correspond to the TCD method fatigue life estimates for connection with a 65 and 140 MPa rivet clamping forces. A clear observation is considerably longer fatigue lives at the rivet hole edge as a result of rivet clamping force increase compared to those obtained for connection with 30 MPa rivet clamping force. On the contrary, the fatigue lives predicted for the angle fillet show only a slight reduction. For instance, in the case of the angle fillet at the top row of the connection for an applied load of \( P = 100 \, \text{kN} \), the VM predicted fatigue lives in the order of \( 2.5, 1.7 \) and \( 1.1 \times 10^6 \) cycles for rivet clamping forces of 30, 65 and 140 MPa, respectively. This corresponds to a fatigue life reduction of 33% and 55% as a result of a rivet clamping force increase from 30 MPa to 65 and to 140 MPa, respectively. This was explained earlier in the results presented in Figure 8-18 (b), in which an increase in the rivet clamping force from 30 to 140 MPa led to an increase in compressive stresses in the angle material under the rivet head as well as an increase in the flexural stresses at the angle fillet region.

Increase in the rivet clamping force is also seen to have a noticeable effect on the fatigue life predictions at the rivet shank to head junction at both the top and second top rivet rows. As can be seen in Figure 8-24 (c), Figure 8-24 (f), Figure 8-25 (i), Figure 8-25 (m), Figure 8-26 (q)
and Figure 8-26 (u), it appears that the increase in rivet clamping force results in slightly higher fatigue life predictions for the rivet shank to head junction for higher load levels while the fatigue life at lower load levels (i.e., \( P = 30, 40 \) and \( 50 \) kN) experiences significant reduction. Figure 8-25 (h) presents the TCD estimates at the top row rivet hole edge for the connection with a 65 MPa clamping force. An interesting phenomenon that is captured in the results shown in this figure is that, the fatigue life predictions for different formalisations of the TCD are slightly scattered at lower applied loads while for applied loads, \( P \), equal to or larger than \( 80 \) kN, the predictions become effectively identical. The same trend can be seen when looking at the results in Figure 8-25 (P) in the fatigue life estimates for the top row rivet hole edge for the connection with a 140 MPa clamping force. In this instance, the scatter disappears for \( P \) values equal to or greater than about \( 150 \) kN. In contrast, this feature did not occur in the predictions for the top row rivet hole edge for the connection with a 30 MPa rivet clamping force, as shown in Figure 8-24 (b). This could be explained by examining the ‘load level’ at which the backside of the outstanding leg of the angle at the location under the rivet head became separated from the floor-beam web for models with 30, 65 and 140 MPa rivet clamping forces. As shown in Figure 8-17, an increase in rivet clamping force causes ‘slightly’ stiffer behaviour in the connection response and restriction of the angle deformation near the rivet. This is because higher axial stiffness in the rivets as compared to the bending stiffness of the outstanding leg of the connection angle. In the FE analysis of the stringer-to-floor-beam connection by Al-Emrani [84], the separation of the angles’ backside from the floor-beam web occurred at \( P \) equals to 30, 80 and 170 kN for connections with rivet clamping force of 30, 65 and 140 MPa, respectively. In this study, this separation occurred at \( P \) values of 30, 80 and about \( 150 \) kN for connections with rivet clamping forces of 30, 65 and 140 MPa, respectively. Such separation eliminates the contact pressures due to friction at the back of the angle, and therefore, stress field becomes more linear and purely due to bending of the angle leg. This explains the scatter observed in the fatigue life estimates for the rivet hole edge at the top two rows for the lower loads and fairly identical results at higher applied loads.

The S-N approach’s fatigue life predictions are also shown in Figure 8-24, Figure 8-25 and Figure 8-26 for connections with 30, 65 and 140 MPa rivet clamping force, respectively.
The results of the above figures show that, in the case of the angle fillet and rivet hole edge, since the nominal stress range obtained from the FE analyses correspond to the same stress readout point (see Figure 8-23), the fatigue lives predicted for these hot-spot locations are effectively identical. Moreover, as can be observed in Figure 8-24 (a–b) and Figure 8-24 (d–e), Figure 8-25 (g–h) and Figure 8-25 (k–l), Figure 8-26 (n–p) and Figure 8-26 (s–t), very similar fatigue lives were estimated at the angle fillet and rivet hole edge for connections with 30, 65 and 140 MPa rivet clamping force subjected to a given applied load, \( P \). This indicates that the S-N method was not capable of successfully capturing the effects of increasing the rivet clamping force on predicted fatigue lives for these hot-spot locations. This is despite the fact that the FE analysis results demonstrated that the stress state in the vicinity of the angle fillet and rivet hole edge changes slightly with increasing rivet clamping forces (see Figure 8-18). These effects were successfully captured by the TCD method predictions.

The results in Figure 8-24, Figure 8-25 and Figure 8-26 clearly show that the S-N method significantly overestimates the fatigue life for the investigated hot-spot locations. For instance, for the angle fillet location in the connection with a 30 MPa rivet clamping force, fatigue lives 20 to 42 times greater were predicted using the S-N method compared to the VM estimates. For the same hot-spot location in the connection with a 140 MPa rivet clamping force, the overestimation of fatigue life was between 36 to over 130 times at different load levels as compared to the corresponding results obtained using the VM.

The S-N method was seen to be capable of capturing the effects of the rivet clamping force increase in the fatigue life predictions at the rivet shank to head junction. This is because a rivet clamping force increase resulted in a reduction in nominal stresses in the rivets which were used in fatigue life estimation according to the S-N method. Despite that, considerable overestimation of fatigue life was seen in the S-N method predictions for the rivet shank to head junction. As Figure 8-24 (c) depicts, the S-N method estimates were between 40 to 2,900 times greater compared to the corresponding VM predictions for the rivet shank to head junction at the top rivet row of the connection with a 30 MPa clamping force.

Comparison of the predictions with the experimental data also shows substantial overestimation in the S-N method fatigue life estimates. According to the experimental test results on stringer-to-floor-beam connections subjected to an applied load of \( P = 100 \text{kN} \) in four-point bending, fatigue life of between \( 0.15 \times 10^6 \) to \( 0.9 \times 10^6 \) cycles was observed for rivets fastening the outstanding leg of the connection angle to the floor-beam [12]. According to
the results in Figure 8-24 (c) for the connection with a 30 MPa rivet clamping force, the least overestimated prediction of the S-N method (i.e., Detail Category 71 fatigue life predictions) compared to the fatigue life obtained in the experimental tests was between 95 to 571 times greater. In contrast, the corresponding predictions of the TCD method were seen to fall within the results of the experimental tests (see Figure 8-24 (c)).

Table 8-2 provides details on factors by which the S-N method predictions were seen to be overestimating the fatigue life as compared to the TCD estimates for a stringer-to-floor-beam connection with a 30 MPa rivet clamping force at the top rivet level hot spots. To achieve more realistic predictions when S-N method is devised for fatigue life assessment of such connections in railway bridges, it is possible to adopt the factors provided in the above table in conjunction with the nominal stresses obtained from a global-local model of the considered detail. However, in most of the cases, because of simplicity, global analyses are used for fatigue assessment of critical details in riveted railway bridges. Research has shown that the S-N method predictions when using the nominal stress obtained from the global model can lead to nonconservative results by a large factor [11]. Therefore, bearing in mind the results of the previous research and in light of the outcomes achieved in this study, the TCD method, despite being slightly time-consuming because of model development and analysis, is recommended to be used when assessing the fatigue life of a complex riveted detail. The primary reason is that, a global-local model can more realistically take into account the effects of the main factors influencing the fatigue performance of the connection. Effects of factors such as connection stiffness, rivet clamping force, mechanical imperfections, different connection geometries and many others can be easily investigated with regard to fatigue life of complex riveted details in railway bridges.

Table 8-2  Comparison of fatigue life predicted using the TCD and the S-N methods for mild steel stringer-to-floor-beam connection with a 30 MPa rivet clamping force at the top rivet level hot-spots

<table>
<thead>
<tr>
<th>Condition</th>
<th>Detail Category 71</th>
<th>Class D</th>
<th>Modified Class B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle Fillet</td>
<td>20-42</td>
<td>43-90</td>
<td>17-90</td>
</tr>
<tr>
<td>Rivet Hole Edge</td>
<td>30-60</td>
<td>61-126</td>
<td>19-163</td>
</tr>
<tr>
<td>Rivet Shank-Head Junction</td>
<td>84-490</td>
<td>178-1037</td>
<td>40-2900</td>
</tr>
</tbody>
</table>
8.6 Concluding remarks

In this chapter, all different formalisations of the TCD were used to estimate the fatigue life of the stringer-to-floor-beam connection of the Vindelälven railway bridge subjected to four-point bending at various load levels. Three different rivet clamping force values of 30, 65 and 140 MPa were used in the global-local FE analyses to assess their effects on the connection behaviour and fatigue life estimates. Fatigue life was also assessed by adopting the mean S-N curves of BS 5400 Class D, Eurocodes Detail Category 71 and BS 5400 Modified Class B in conjunction with the nominal stress range obtained from the FE analysis. The angle fillet and rivet hole edge in the outstanding leg of the angle as well as the rivet shank to head junction for the rivets connecting the angle leg to the floor-beam web at the top and second top rivet row levels were considered as the fatigue-critical hot-spots in the fatigue life prediction process.

The FE-estimated rotational stiffness of the connection with a 30 MPa rivet clamping force was found to be in the order of $2.5 \times 10^5$ kNm/Rad and in good agreement with the corresponding values obtained for the tested specimens. The TCD method was found to be very successful in predicting fatigue life of the stringer-to-floor-beam connection when compared to the corresponding experimental observations. The predictions of the PM, LM, AM and VM were seen to fall within the scatter of the experimental data proving the fatigue life prediction capabilities of the TCD method when used to assess complex riveted details. The rivet clamping force increase was found to have fatigue life-reducing effects at the angle fillet at both the top and second top rivet row. On the contrary, the fatigue life at the rivet hole edge hot-spot was improved as a result of rivet clamping force increase. The connection stiffness was also seen to only marginally rise with the increase in the rivet clamping force resulting in the development of slightly greater stringer-end moments in the connection angle.

On the other hand, the S-N method predictions were seen to be very optimistic. Eurocodes Detail Category 71 was found to be the least nonconservative S-N classification. Despite that, fatigue life predictions of the Detail Category 71 were found to be optimistic by a factor of 20 to 42, 43 to 90 and 17 to 90 for the angle fillet, rivet hole edge and rivet shank to head junction, respectively, at the top rivet row of the connection with a 30 MPa rivet clamping force subjected to four-point bending. The S-N method was found to be unable to capture the effect
Fatigue life evaluation of riveted stringer-to-floor-beam connections

of the rivet clamping force increase on the fatigue life of the angle filet and rivet hole edge. This phenomenon was successfully captured in the predictions of the TCD method.
9 Conclusions and future work recommendations

9.1 Conclusions

The main objective of this thesis was to assess the capabilities and applicability of the Theory of Critical Distance method (TCD) for estimating fatigue life of complex riveted bridge details. An accurate and reliable fatigue life estimate was defined as predictions falling inside the scatter of the experimental data or calculated fatigue strength values with less than about 20% deviation from the mean curve for the test results. In this study, the TCD method’s fatigue life predictions were compared to those estimated using the S-N approach to reach a better understanding of the implementation and limitations of both these methodologies.

The first step (Chapter 3) in the adopted scientific approach to achieve the thesis objectives was the identification and presentation of the S-N curves suggested by British and European Codes of Practice for fatigue assessment of riveted bridge details. Additionally, different formalisations of the TCD method as used for fatigue assessment of riveted details were also described. Additionally, the relationship between critical length and the number of cycles to failure, $L$ vs. $N_f$, required for fatigue assessment of old metallic riveted details using the TCD method was determined in this step.

A database of fatigue tests on fatigue-prone riveted bridge details was obtained from the technical literature. A brief description of some of these tests and their outcomes were presented in the second step of the approach (Chapter 4). In the third step, to identify suitable element types and degree of mesh refinement for the finite element analysis (FEA), benchmark FE analyses were performed on simple models of riveted single lap joint (Chapter 5). The benchmark FE analyses were aimed to increase confidence in the modelling techniques adopted for the development of the FE models of more complex details considered in this study for fatigue assessment based on the TCD and S-N approaches. Fatigue life assessment of structural wrought-iron and mild steel simple details such as, plates with one or multiple circular rivet holes and single, double and butt joints, based on both the TCD and the S-N methods was performed in the fourth step (Chapter 6). Following that, the fatigue analysis of wrought-iron and mild steel built-up girders of old riveted railway bridges was carried out based on the TCD and the S-N methods, and the results were compared to the corresponding experimental data (Chapter 7).
Quantification of the differences between the S-N and the TCD fatigue life predictions compared to the experimental data for the simple details and built-up girders was used as a measure to assess the predictive capabilities of the S-N and the TCD methods. The final step of the approach was the fatigue analysis of stringer-to-floor-beam connections, identified as one of the most fatigue-prone details in riveted railway bridges, through a global-local FE analysis (Chapter 8).

Majority of the available fatigue related research are about primary members of old riveted bridges (for instance, cross-girders or stringers) owing to the crucial load-carrying role played by these elements. However, experience and research have shown that the stringer-to-floor-beam connections in these old bridges are often more prone to fatigue damage because of the deformation-induced secondary stresses developed in different components of these connections. The S-N method, as the commonly suggested fatigue assessment approach in the British and European Codes of Practice, has been unable to take into account these effects when used for fatigue assessment of such fatigue-critical connections. This thesis is a novel attempt to verify and quantify the predictive capability of the TCD method in fatigue life assessment of such connections in old metallic riveted railway bridges. Thus, a finite element global-local model of a stringer-to-floor-beam connection obtained in the literature survey was developed. The results of the FE analyses of the connection were used for fatigue life prediction on the basis of the TCD and S-N method. The primary findings and principal conclusions of each chapter is summarised below in a chronological order.

In Chapter 3, as the first step of the scientific approach, through a literature survey, the S-N curves suggested by BS 5400-10, BS EN 1993-1-9 and the Railtrack Code for fatigue assessment of riveted details were identified. The mean S-N curve of BS5400 ‘Class D’ and ‘Modified Class B’, BS EN 1993 ‘Detail Category 71’, Railtrack Code ‘Class WI-rivet’ (related to wrought-iron riveted details) were the detail classifications considered in this research. The TCD method was also explained in this chapter. The $L$ vs $N_f$ relationship required for the TCD method fatigue assessment calibrated and determined for the wrought-iron material of the Chepstow Bridge.

*Modified Class B and Class WI-rivet were found to be effectively identical and likely to result in very similar fatigue life predictions. The $L$ vs. $N_f$ relationship determined in Chapter 3 revealed that the critical length, $L$, for old structural wrought-iron take a value of about 1.7*
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The $L$ value increases for a smaller number of cycles to failure in the medium-cycle fatigue region.

In Chapter 4, some of the fatigue tests on riveted bridge members and connections obtained in the literature were summarised.

Study of the collected fatigue test database identified the stringer-to-floor-beam connections as one of the most fatigue-critical details commonly encountered in riveted railway bridges. These connection types are subjected to unforeseen secondary deformation-induced flexural stresses during their service life.

A benchmark FE analysis was performed on a single-rivet single lap joint in Chapter 5. The main aim of this parametric study was to deduce which element type and mesh density combination is capable of adequately capturing the stress variation through the plate thickness in SLJ specimens subjected to longitudinal uniaxial tensile loading.

The study showed that fully- or reduced integration 3-D 20-noded solid elements (C3D20R or C3D20) with at least 4 layers of elements through the plate thickness and a minimum of 24 elements around the rivet shank/rivet hole perimeter are adequate to accurately capture the stress gradient at the vicinity of the stress concentration. The accuracy of the 8-noded solid elements in determining the distribution of the linear-elastic stress field at the rivet hole edge was found to be insufficient irrespective of the mesh refinement used.

Overall, confidence was gained by the results of the benchmark study to apply the finite element technique in modelling simple and complex details for fatigue life assessment in Chapters 6, 7 and 8.

In Chapter 6, the fatigue life of some of the simple details obtained through the literature survey was assessed based on the TCD and the S-N methods. The investigated details included wrought-iron flat plates with one, two or six circular hole(s), mild steel single and double lap joints and wrought-iron butt joint. The $L$ vs. $N_f$ relationship determined in Chapter 3 was adopted for fatigue life predictions based on different formalisations of the TCD method, that is, the PM, LM, AM and VM. Fatigue life prediction according to the S-N approach was also carried out by using the different S-N classifications identified in Chapter 3 for riveted details. The nominal stress was obtained from the results of the FE analysis. In the case of riveted joints, different rivet clamping force values were used in the analysis to assess the predictive...
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capabilities of the applied assessment methods. The S-N and the TCD method fatigue life predictions were compared with the experimental data using accuracy diagrams with estimated fatigue life, $N_{f,e}$, and experimentally obtained fatigue life, $N_f$, plotted on the X- and the Y-axis, respectively.

The TCD method was seen to be successful in predicting fatigue life with predictions in most of the cases falling within the constant amplitude scatter bands of the experimental data and consistently more accurate than estimates based on the S-N approach. In majority of the cases, the PM, AM and VM predictions appeared to be very similar. In the case of the mild steel single lap joint (SLJ), the TCD method was capable of accurately estimating the rivet clamping force values developed in the rivets of the specimens tested in the experimental investigation, conforming well with the experimental data. On the contrary, the fatigue life estimations of the S-N approach for the rivet hole edge at the inner face between the two plates were seen to be conservative in the high-cycle region and effectively unaffected when rivet clamping force was increased from 2 to 22 and 40 MPa. This indicates that the S-N method was unable to capture the effect of the rivet clamping force increase on the fatigue life of the connection. The fatigue life estimates were seen to be considerably sensitive to the choice of the detail classification. Predictions of Detail Class D were the least conservative (up to a factor of 2 in the high-cycle region) compared to other S-N classifications adopted.

The wrought-iron butt joint investigated in this study included specimens with 5 or 8 mm thick plates. In the case of the wrought-iron butt joints with 8 mm thick plates, the TCD method estimated rivet clamping force values of about 30 to 40 MPa in the rivets of this connection type while this value was about 20 MPa for the specimens with 5 mm thick plates. This was found to conform well with the findings of other research showing rivets with longer grip length are capable of developing higher rivet clamping forces. Predictions of the PM, AM and VM were seen to be very similar. The accuracy of the S-N method predictions was low and mostly conservative and outside the CA scatter bands of the experimental data. The sensitivity of the fatigue life estimates to the detail classification was seen to be considerable with Detail Class D predictions being the least conservative (up to a factor of 6 in the high-cycle region) compared to the other S-N classifications adopted.

The mild steel double lap joint case study considered in this thesis contained specimens with two different rivet clamping force values of 76 and 253 MPa. In both cases, the TCD predictions
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were seen to always fall inside the CA scatter bands with maximum fatigue strength error interval of ±13%, which is well within the 20% target error factor. In comparison, the fatigue life predictions of the traditional S-N method were seen to suffer with slight degree of conservatism for specimens with 253 MPa clamping forces. Only predictions of Class D in the medium-cycle region were seen to fall inside the error bands. For specimens with a 76 MPa clamping forces most of the fatigue life predictions were within the CA scatter bands and in line with the TCD predictions. The sensitivity of the fatigue life predictions to the choice of detail classification were seen to be substantial with Modified Class B estimates being considerably more conservative (up to a factor of 5 in the high-cycle region) when compared to Detail Class D predictions.

Overall, the use of the $L$ vs. $N_f$ relationship calibrated using the wrought-iron material fatigue data of the Chepstow Bridge was not seen to have significant adverse effects on the accuracy of the TCD predictions when it was used to assess fatigue life of various simple details in Chapter 6.

Having gained confidence in the predictive capabilities of the TCD method in Chapter 6, through the fatigue assessment of simple details, the focus on Chapter 7 was to assess the applicability of the TCD method when used to assess the fatigue life of more complex details such as riveted built-up bridge girders. Five different case studies were investigated by adopting the TCD method as well as the S-N approach for fatigue life assessment of riveted railway bridge girders subjected to three- or four-point bending fatigue loading at different stress ratios. Some of the girders were salvaged from dismantled bridges after reaching their service life and some were in an unused condition. The submodelling technique was adopted for modelling the global-local FE models of the built-up girders to reduce computational costs. Fatigue life predictions of the TCD and the S-N method were compared to the corresponding experimental data to form accuracy diagrams.

The novel formalisations of TCD were seen to be extremely successful in predicting the fatigue life of full-scale riveted built-up girders subjected to bending stresses. The level of accuracy observed in the TCD predictions for full-scale girders was higher than those seen when TCD was applied for fatigue assessment of flat plates with circular holes presented in Chapter 6.

The TCD predictions were also seen to have higher accuracy and consistency as compared to estimates of the S-N approach. The results of the TCD method in medium- and high-cycle
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Fatigue were seen to always fall inside the CA scatter bands of the experimental data while the S-N approach predictions were mostly on the conservative side and in some instances outside the CA scatter bands. Fatigue life estimates were seen to be very sensitive to the S-N classification with Class D being the only detail classification of the S-N approach capable of predictions, despite being close to the conservative extreme of CA scatter bands but in most of the cases falling inside the scatter bands. The predictions of Class D were the least conservative compared to those of other investigated detail classifications up to a factor of 5 in the high-cycle fatigue region.

For the case studies where the material-specific plain S-N curve for the girder material was not available to utilise in TCD fatigue assessment, a plain-material S-N curve obtained from the literature for the same material type generated at the same stress ratio as the one damaging the investigated detail was used for TCD fatigue assessment. The fatigue life predictions of the TCD method in such cases were seen to be slightly sensitive to the choice of the plain S-N curve but still an acceptable level of accuracy was attained.

In general, it was confirmed that adopting the L vs. $N_f$ relationship calibrated for wrought-iron material of Chepstow Bridge can still lead to highly accurate predictions when the TCD method is used for fatigue assessment of wrought-iron or mild steel riveted built-up girders.

The TCD method was capable of estimating the rivet clamping forces present in the rivets of the girders investigated in the experimental research. This was confirmed by comparing the rivet clamping force estimated by the TCD method against the rivet clamping force experimentally measured for the Vindelälven Bridge built-up stringer as well as Hinterrhein Bridge tapered built-up girders.

Overall, the results of Chapter 7 provided verification on applicability and predictive capabilities of the TCD method when used to assess the fatigue life of more complex riveted bridge details.

The fatigue performance of the stringer-to-floor-beam connection of the Vindelälven Bridge was investigated in Chapter 8. The 3-D finite element model of the double-angle connection was created using the modelling techniques verified in Chapter 5. The local behaviour of the connection under four-point bending was compared to the experimental data to verify the rotational stiffness developed in the FE model. Fatigue analyses were carried out based on
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the TCD and the S-N method to predict the fatigue life of various components of the connection at the level of the top and second top rivet rows. The angle fillet, rivet hole edge in the outstanding leg of the angle and the rivet shank to head junction in the rivets connecting the angle leg to the floor-beam web were considered as the main hot-spot locations for fatigue life prediction. Three different rivet clamping forces were considered in the FE analysis to investigate their effects on the fatigue performance of the stringer-to-floor-beam connection.

Good correlation was observed between the stresses calculated through the FE analysis compared to that observed in the tested ‘specimen I’ with only about 8% difference in the magnitude of peak stresses.

The FE model with 30 MPa rivet clamping force experienced out-of-plane deformations in the order of the magnitudes of those observed in the experiments. A good agreement was also found when comparing the FE-calculated nominal stresses in the top rivet of the model with a 30 MPa clamping force to the corresponding experimentally measured data for ‘specimen I’. This indicated that rivet clamping force magnitudes of about 30 MPa were present in the rivets of the tested stringer-to-floor-beam specimens.

Increasing the rivet clamping force from 30 to 140 MPa was seen to improve the connection stiffness by about 19% which corresponds to a 7% growth in the stringer-end moment. Such an increase in the rivet clamping forces also resulted in a 15% average reduction in the deformation induced by the external applied load at the top of the angle leg. Higher rivet clamping force values were seen to create a state of close to partial fixity at the rivet centreline of the outstanding leg, causing the connection to behave more in line with that suggested by the Wilson model, which could lead to greater compressive stresses in the angle material under the rivet head as well as an increase of the flexural stresses at the fillet.

Overall, the degree of continuity and rotational stiffness displayed by the FE models were in good agreement with the range of stiffness values observed for tested ‘specimens I, II and III’.

All formalisations of the TCD method were adopted to predict fatigue life of the stringer-to-floor-beam connection with three different rivet clamping forces of 30, 65 and 140 MPa. The \( L \) vs. \( N_f \) relationship determined for wrought-iron material of the Chepstow Bridge was used
to calculate the critical distance required for fatigue life estimation. The fatigue life predictions were presented in the form of stress vs. number of cycles to failure (S-N curves).

The TCD was seen to be successful in ranking the most critical hot-spot location, that is, the rivet shank to head junction, angle fillet and rivet hole edge, respectively, when compared to the test results. The TCD-predicted fatigue life for the rivet shank to head junction for the top rivet of the connection with a 30 MPa rivet clamping force subjected to an applied load of, $P = 100 \text{ kN}$, correlated well with the results of the experimental investigation, confirming a high level of accuracy in the TCD predictions.

The increase in rivet clamping force from 30 to 65 and 140 MPa were seen to result in greater fatigue life estimates at the rivet hole edge by up to a factor of 10 to 22 times, respectively. On the contrary, such rivet clamping force increase only caused a reduction in estimated fatigue life at the angle fillet by a factor of up to 1.5 and 2.5 for connections with 65 and 140 MPa rivet clamping force values, respectively. In the case of the rivet shank to head junction, it was seen that the increase in rivet clamping force resulted in slightly higher fatigue life predictions for the rivet shank to head junction for higher load levels while the fatigue life at lower load levels (i.e., $P = 30, 40$ and 50 kN) experienced notable reduction.

Increase in rivet clamping force caused slightly stiffer behaviour in the connection response and restriction of the angle deformation near the rivet. Because of higher axial stiffness of the rivet compared to the bending stiffness of the outstanding leg of the angle, the separation of the angle back side from the floor-beam web occurred at load values of 30, 80 and about 150 kN for connections with rivet clamping force values of 30, 65 and 140 MPa, respectively. This was in very good agreement with the values observed in a different FE analysis study performed on the same connection. This phenomenon was captured in the predictions of the TCD method.

Fatigue assessment of the stringer-to-floor-beam connection was also carried out based on the S-N approach using Class D, Detail Category 71, and modified Class B detail classifications. The nominal stresses required for fatigue life estimation were obtained from the results of the FE analyses of the stringer-to-floor-beam connection. The same hot-spot locations as those considered for TCD fatigue assessment were also used for S-N method fatigue life estimation.
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The S-N method’s fatigue life predictions for the angle fillet and rivet hole edge at each rivet level were seen to be effectively identical since the same nominal stress range value was used for both these hot-spot locations. The S-N method was deemed unable to capture the effects of increasing the rivet clamping force on fatigue life at the rivet hole edge as well as the angle fillet.

The S-N method’s fatigue life estimates were compared to the available experimental data as well as the predictions of the TCD method.

It was found that the S-N method is considerably nonconservative in predicting the fatigue life of a stringer-to-floor-beam connection in a railway bridge (up to a factor of 2,900). The predictions of the S-N method were seen to be sensitive to the choice of the detail classification. It was proposed that the S-N method can still be used for fatigue life assessment of the stringer-to-floor-beam connections in conjunction with the pertaining reduction factor recommended in Chapter 8 for different detail classification and hot-spot locations.

Overall, the TCD method was found to be very successful when used to assess the fatigue life of complex riveted details subjected to complex fatigue loading. The only downside to the use of the TCD method would be the complex and time-consuming detailed local modelling stage and analysis time. The high analysis time is mainly because of the expensive 3-D 20-noded elements and the refined mesh used in the modelling of the connection. However, the amount of time spent could be justified by the level of accuracy obtained when the TCD method is used compared to unrealistic outcome achieved when using the S-N method. Moreover, since the results obtained using the Point Method (PM) or Area Method (AM) is not significantly different from the more time-consuming Volume Method (VM), the author recommends using the former to reduce the post processing time (i.e, the PM or AM).
9.2 Recommendations for future work

9.2.1 Experimental research

The application of the TCD method for fatigue assessment of a detail uses a material-specific $L$ vs. $N_f$ relationship. Such a relationship is determined according to the procedure described in Figure 3-2. It uses the $S$-$N$ curves generated under constant amplitude fatigue tests on the plain and notched specimens made from the material of the investigated detail. A known geometrical feature as sharp as possible is recommended to be employed. In this thesis, because of a lack of a detail-specific $L$ vs. $N_f$ relationship, in all the case studies the $L$ vs. $N_f$ relationship calibrated for the Chepstow Bridge wrought-iron material was adopted for the TCD method fatigue life assessment. More fatigue tests should be carried out on structural mild steel to allow calibration of material-specific $L$ vs $N_f$ relationship to be used for fatigue life assessment of the pertaining case studies assessed in this thesis. Despite achieving a high level of accuracy in the TCD estimated fatigue life in the current study, it would be insightful to quantify the differences in the predicted fatigue life when a mild steel-specific $L$ vs $N_f$ relationship is used in conjunction with the TCD method.

The value of critical length, $L$, in the high-cycle region can be calculated through Eq. 2.4 by using the range of threshold stress intensity factor, $\Delta K_{th}$, and plain material fatigue limit, $\Delta \sigma_o$, both under the same stress ratio, $R$. A thorough literature survey of mechanical and fatigue properties of structural wrought-iron or mild steel revealed that there is a lack of adequate knowledge on $\Delta K_{th}$ and plain material $\Delta \sigma_o$ for the same $R$ ratio. Additional tests could be carried out on the available structural wrought-iron and mild steel material to fill up the knowledge gap for stress ratios encountered in practical situations. The results of these tests could help in confirming the results of $L$ vs. $N_f$ relationship in the high-cycle region and consequently the TCD fatigue predictions.

Similar tests could also be carried out to calibrate a new $L$ vs. $N_f$ relationship for structural wrought-iron material from a different source to assess the sensitivity of the fatigue life predictions to the choice of such relationship.

In the case of the stringer-to-floor-beam connections in riveted bridges, the amount of available full-scale tests on fatigue performance of such connections is very limited. More fatigue tests under different stress levels are needed to better understand the behaviour of
these connections. It would also be beneficial towards investigating fatigue crack initiation and propagation in different components of these connections. The test results could help develop fatigue S-N curves that would better represent stringer-to-floor-beam connection details since the secondary effects would be explicitly included.

9.2.2 Analytical research

In this thesis, the fatigue assessment based on the TCD method was performed based only on constant amplitude (CA) fatigue loading. Additional finite element analysis of the connection investigated in Chapter 8 under variable amplitude (VA) loading is needed. This would lead to verification of the TCD method fatigue life predictions under more realistic loading conditions. Different traffic mixes of BS EN 1991-2:2003 or BS 5400-10 could be used for VA fatigue loading of the connection to create a load spectrum to which a typical stringer-to-floor-beam connection of an old metallic riveted railway would be subjected. Additionally, to create a loading condition more representative of practical situations, the effects of the rails, ballast and sleepers on spreading the axle loads on the stringers could also be included in the finite element analyses of the connections subjected to VA loading. This could assist in quantifying the differences in the TCD fatigue life estimates to a case where no load spread is considered in the analysis.

Experimental investigations on stringer-to-floor-beam connections has shown that fatigue failure of the rivets (rivet heads popping off) in the top two rows was enough to substantially reduce the propagation rate of existing fatigue cracks in the angle leg at the fillet. Finite element analysis of the same connection investigated in Chapter 8 could be carried out to assess the effect of various assumed mechanical defects such as empty top rivet holes, top rivets with no heads, misalignment of top rivets, smaller rivet head in the top rivet, and so on, on fatigue life predictions. The TCD method could be adopted to perform fatigue life estimates for all different scenarios to better capture the effects of such defects on fatigue performance of double-angle riveted connections.

As previous research has shown, gauge distance is one of the main factors affecting the flexural flexibility and in turn the fatigue performance of the double-angle riveted connections. Therefore, finite element analysis could be performed for stringer-to-floor-
beam connections with different gauge distance values, representative of practical situations, to investigate its effects on the fatigue life of various components of such connections. This could extend the findings of this thesis to a wider range of connection types.

9.2.3 Field measurement

In the previous section, the recommendation was to perform finite element analysis of the stringer-to-floor-beam connection subjected to train traffic loading as suggested in the relevant codes of practice taking into account the load spread effects due to rails, ballasts and sleepers. Field measurements on a stringer-to-floor-beam connection of a typical riveted railway bridge could help verify the results of the FE analyses. The load spectra obtained from field measurement can be compared to that used in the FE analysis to provide valuable information on the dynamic effects of train loading on the load history and its effects on the TCD fatigue life predictions.
Appendix A

Worked example:
Uniaxial fatigue tests on mild steel single lap joint connections (SLJ) made of Trezoi Bridge material at stress ratio, $R$, of +0.1 yielded a mean S-N curve with detail shown in Figure A-1 [75]. A SLJ specimen is subjected to a constant amplitude cyclic load range of $\Delta P_r = 30$ kN ($P_{\min}$ and $P_{\max}$ of 3 and 33 kN, respectively). It is required to calculate the fatigue life of this SLJ specimen using the Theory of Critical Distances (TCD) and compare with the experimentally determined mean S-N curve. The dimensions of the SLJs are also given in Figure A-2.

![Figure A-1. Mean S-N curve of the tested SLJ specimens under uniaxial fatigue loading (gross stress S-N curve) [75]. BS 5400-10 Class B mean S-N curve is used to represent plain mild steel material [28].](image)

**Step 1.** Determination of the critical length vs. number of cycles to failure ($L$ vs. $N$):
In majority of the practical situations for fatigue assessment, the required data for determination of the $L$ vs. $N$ relationship is not readily available (plain and notched material S-N curves for the investigated material under fully reversed stress ratio).
Figure A-2. Mild steel single lap joint subjected to uniaxial tensile force ‘ΔP,’ at load ratio, R, of +0.1.

To show the procedure adopted in such conditions, this worked example adopts a procedure that can be applied in such practical situations. However, the procedure used to determine the $L$ vs. $N_f$ relationship is also briefly explained.

Determination of a material-specific $L$ vs. $N_f$ relationship requires the S-N curves of the plain and a known notched specimen made of the same material to be available. Both these S-N curves shall be obtained for stress ratio, $R$, of -1 (representing fully reversed uniaxial fatigue tests). Having obtained the two S-N curves, the $L(M)$ vs. $N_f$ relationship in the medium cycle region can be calculated using the procedure described in Figure A-3. Repeating the above procedure for several number of cycles to failure values leads to calibration of the $L(M)$ vs. $N_f$ relationship. It should be noted that the principal stress amplitude vs. distance curve at the notch root can be determined using either analytical or numerical tools.

As discussed earlier, if such data is not available, the $L$ vs. $N_f$ relationship obtained for structural wrought-iron material of the Chepstow Bridge can be used (see Figure A-4).
Figure A-3. Calibration of the $L$ vs. $N_f$ relationship in medium-cycle fatigue according to the Point Method using plain and notched S-N curves and the linear-elastic stress field ahead of the notch.

Figure A-4. The $L$ vs. $N_f$ relationship for the Chepstow Bridge wrought-iron material [38] (representing old structural metal). Blue and orange colours refer to medium- and high-cycle regions, respectively.
Step 2. Calculation of the principal stress vs. distance curve at the critical hot-spot:

The single lap joint (SLJ) rivet hole edge and the rivet shank to head junction are the likely hot-spot locations. It is convenient to obtain the principal stress vs. distance curve by using finite element analysis. The FE model would be subjected to a pressure of

\[ \Delta \sigma = \frac{\Delta P_r}{b \times t} = \frac{30 \times 1000}{60 \times 10} = 50 \, \text{MPa} \]

Using FE analysis of the SLJ with a 40 MPa rivet clamping force and subjected to a tensile pressure of 50 MPa, the principal stress vs. distance curve shown in Figure A-4 was obtained:

![Figure A-5: Principal stress vs. distance curve at the rivet hole edge for SLJ with a 40 MPa clamping force and subjected to \( \Delta P_r = 30 \, \text{kN} \).](image)

Step 3. Calculation of fatigue life using the TCD:

Imagine the S-N curve corresponding to the tested SLJ specimens is not known, as is the case in most practical situations. Therefore, the number of cycles to failure \( N_{f,e} \) corresponding to the gross applied stress of \( \Delta \sigma = 50 \, \text{MPa} \) cannot be known. At this point, it is possible to use a trial-and-error method to estimate the \( N_{f,e} \) value.
For this initial $N_{f,e}$ value, the corresponding stress range, $\Delta \sigma_{S-N}$, on the plain material $S-N$ curve can be calculated using the expression of the $S-N$ curve:

$$\Delta \sigma_{S-N} = \sigma_A \sqrt[2]{\frac{N_A}{N_{f,e}}}$$  \hspace{1cm} \text{Eq. A-1}$$

where $m$ and $N_A$ are 3 and $2 \times 10^6$, respectively, for BS 5400-10 Class B mean $S-N$ curve and $\sigma_A$ is the value of stress at $N_A$ (185 MPa).

The value of the critical length, $L$, corresponding to the $N_{f,e}$ value can also be obtained using the appropriate relationship given in Figure A-4. The $L$ value can then be used in conjunction with the principal stress vs. distance curve at the critical hot spot, shown in Figure A-5, to determine the amplitude of a reference stress, $\sigma_{ref}$. The estimated number of cycle to failure for the SLJ based on the Point method (PM) would be the $N_{f,e}$ value in which the $\Delta \sigma_{S-N}$ and $\sigma_{ref}$ become equal. Therefore, according to the PM method, the SLJ subjected to a cyclic load range of $\Delta P = 30$ kN would have an $N_{f,e}$ value of $1.5 \times 10^6$ cycles (see Table A-1).

<table>
<thead>
<tr>
<th>No.</th>
<th>$N_{f,e}$</th>
<th>$\Delta \sigma_{S-N}$</th>
<th>$L$</th>
<th>$L/2$</th>
<th>$\sigma_{ref}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.00E+06</td>
<td>147.1</td>
<td>1.7</td>
<td>0.8</td>
<td>213.7</td>
</tr>
<tr>
<td>2</td>
<td>4.00E+06</td>
<td>155.6</td>
<td>1.8</td>
<td>0.9</td>
<td>211.3</td>
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<td>1.0</td>
<td>208.1</td>
</tr>
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<td>2.2</td>
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<td>202.8</td>
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<td>1.2</td>
<td>198.7</td>
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<tr>
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<td>2.5</td>
<td>1.3</td>
<td>196.5</td>
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<tr>
<td>7</td>
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<td>2.7</td>
<td>1.4</td>
<td>192.2</td>
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</tbody>
</table>

In the following, the fatigue life based on different formalisations of the TCD is presented:
1. The Line Method (LM):

Fatigue life can also be calculated using the LM method by averaging the principal stress values to a distance of $2L$ from the hot spot. Table A-2 shows the calculations carried out to determine the $N_{f,e}$ value for the SLJ subjected to a cyclic load range of $\Delta P_r = 30\, \text{kN}$. The $N_{f,e}$ value was estimated to be $2.3 \times 10^6$ cycles.

### Table A-2. Calculation of $N_{f,e}$ according to the Line Method

<table>
<thead>
<tr>
<th>No.</th>
<th>$N_{f,e}$</th>
<th>$\Delta\sigma_{s-N}$</th>
<th>L</th>
<th>$2L$</th>
<th>$\sigma_{ref}$</th>
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</thead>
<tbody>
<tr>
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<td>1.70</td>
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<td>189.1</td>
</tr>
<tr>
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<td>155.6</td>
<td>1.80</td>
<td>3.61</td>
<td>183.9</td>
</tr>
<tr>
<td>3</td>
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<td>167.2</td>
<td>1.96</td>
<td>3.93</td>
<td>183.5</td>
</tr>
<tr>
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<td>178.6</td>
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<td>4.25</td>
<td>178.8</td>
</tr>
<tr>
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<td>2.00E+06</td>
<td>185.0</td>
<td>2.22</td>
<td>4.43</td>
<td>178.7</td>
</tr>
</tbody>
</table>

2. The Area and Volume Methods:

It is possible to adopt the Area or Volume Method formalisations of the TCD to estimate fatigue life. The procedure is the same but more time-consuming. As shown in Figure A-6, the critical area and volume at the notch tip (rivet hole edge in this case) is a semicircle or a hemispherical shape with a radius of $1.32L$ and $1.54L$, respectively. The surface stresses in this critical semicircle area were averaged to determine $\sigma_{ref}$ value based on the AM. For the Volume Method, the principal stresses on the surface as well as those within the critical volume were obtained from the FE results and averaged to obtain the $\sigma_{ref}$ value.

The calculations presented in Table A-3 and Table A-4 show the fatigue life estimation carried out according to AM and VM, respectively. The overall results obtained based on all four formalisations of the TCD method are also plotted in the graph in Figure A-7. It can be seen that all different formalisations of TCD provided an estimate within the scatter bands of the experimental data with VM giving the most accurate estimate.
Figure A-6. Critical area and volume around the rivet hole edge adopted to read principal stress values from the FE model. (a) Semicircle and (b) semihemisphere used to average principal stresses based on the AM and VM, respectively. Here, $c$ and $d$ get values of $1.32L$ and $1.54L$ for Area and Volume Methods, accordingly.

Table A-3. Calculation of $N_{f,e}$ according to the Area Method

<table>
<thead>
<tr>
<th>No.</th>
<th>$N_{f,e}$</th>
<th>$\Delta \sigma_{s,N}$</th>
<th>$L$</th>
<th>$1.32L$</th>
<th>$\sigma_{ref}$</th>
</tr>
</thead>
<tbody>
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<td>2.92</td>
<td>208.0</td>
</tr>
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<td>200.3</td>
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<tr>
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<td>1.20E+06</td>
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<td>2.58</td>
<td>3.40</td>
<td>197.9</td>
</tr>
</tbody>
</table>

Table A-4. Calculation of $N_{f,e}$ according to the Volume Method

<table>
<thead>
<tr>
<th>No.</th>
<th>$N_{f,e}$</th>
<th>$\Delta \sigma_{s,N}$</th>
<th>$L$</th>
<th>$1.54L$</th>
<th>$\sigma_{ref}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.00E+06</td>
<td>167.2</td>
<td>1.96</td>
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<td>208.9</td>
</tr>
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<td>3.41</td>
<td>184.56</td>
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<td>198.8</td>
<td>2.41</td>
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<td>220.0</td>
<td>2.72</td>
<td>4.19</td>
<td>168.4</td>
</tr>
</tbody>
</table>
Figure A-7. The fatigue life estimates based on different formalisations of the TCD method at the rivet hole edge for an SLJ subjected to a CA cyclic load range of ΔP_r = 30 kN.
References


References


References


References


References


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References


