Fatigue assessment of an existing steel bridge by finite element modelling and field measurements

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Abstract. The evaluation of fatigue life of structural details in metallic bridges is a major challenge for bridge engineers. A reliable and cost-effective approach is essential to ensure appropriate maintenance and management of these structures. Typically, local stresses predicted by a finite element model of the bridge are employed to assess the fatigue life of fatigue-prone details. This paper illustrates an approach for fatigue assessment based on measured data for a connection in an old bascule steel bridge located in Exeter (UK). A finite element model is first developed from the design information. The finite element model of the bridge is calibrated using measured responses from an ambient vibration test. The stress time histories are calculated through dynamic analysis of the updated finite element model. Stress cycles are computed through the rainflow counting algorithm, and the fatigue prone details are evaluated using the standard SN curves approach and the Miner’s rule. Results show that the proposed approach can estimate the fatigue damage of a fatigue prone detail in a structure using measured strain data.

1. INTRODUCTION

During the lifetime of metallic bridges, traffic loads can induce fatigue issues in its structural details. For these structures there are strong motivations to evaluate not only the remaining fatigue life, but also to develop a reliable and cost-effective approach to ensure appropriate maintenance and management. Relating the evaluated fatigue life to stress ranges is a challenging task. The stress range spectra for bridges are strongly site-specific depending upon vehicle types, range of vehicle speeds, road roughness conditions, ambient environment and bridge type [1]. Field measurements are normally used to consider some of these uncertainties. However, in situ monitoring of all the details of a structure is not feasible in practice. An effective Finite Element model (FEM) is therefore normally deployed in the fatigue analysis. Nevertheless, for a numerical model to accurately reproduce the real response of a bridge, it must be calibrated and updated based on experimental data, gathered in situ experimental campaigns. [2] asserted that an effective FEM is usually calibrated using field measurements such as static and dynamic load tests to update parameters of bridge models. According to Friswell & Mottershead as cited in [3] a Finite Element Model calibrated or updated using modal properties identified through
System identification provides are the best tool for understanding and physical interpretation of the causes and effects of the various loading regimes.

This paper presents the dynamic behavior of a numerical model for the case of a bridge for the passage of roadways traffic, the Bascule Bridge, which is located at the A379 across the Exeter Canal of the UK roadways. The results of a preliminary experimental campaign are also presented, which considered ambient vibration data obtained from real traffic. The results of the experimental work were used for the updating and validation of the numerical model of the bridge. Finally, some of the results of the fatigue damage analysis performed on the structure were presented, according to the BS NA EN 1993 1-9 [4].

2. Case study

A roadway bridge, the Bascule Exeter Bridge, is used as a case study for fatigue evaluation. The fatigue critical connections examined in this study were identified by a preliminary fatigue assessment based on the nominal stress method.

2.1. Bascule Bridge

The Bascule Bridge was built in 1972. It takes the A379 across the Exeter Canal. It is designed to be opened to allow shipping to pass up and down the canal. The bridge may not often open for water-borne, but they are kept busy with vehicles passing over it along the A379. Figure 1 shows overview of the bridge (right) and bridge location.

![Figure 1](image)

(a) Bascule Bridge Location and (b) Bridge overview

2.2. Geometry

The Bascule Bridge is a single span structure with a light-weight aluminium deck supported on cross beams which are in turn supported by two steel longitudinal girders (W36x12 sections) that are simply supported as illustrated in Figure 2.
The lifting section is 17.28m long with a carriageway width of 6.7m and a footway on the outside edge of the parapet of 2m. The width of the bridge is 8.116 m, a distance spanned by a series of transverse cross beams (W21x8.25 sections), that are spaced 0.97 m apart as showed in Figure 3. The deck of this bridge consists of two universal beams with cross beam supporting an aluminium deck planks Figure 3. The aluminium planks are held down onto steel cross beams with bolts Fig. 4. The cross beams are bolted to the web stiffeners on the main beam.
3. The numerical model

In this section, the finite element (FE) which is representative of a common number of short-span, riveted and welded, roadway bridges around the UK. Three-dimensional 3D 8-noded shell elements and were used to model the cross beams, main girders, stiffeners. Solid element 3D 8-noded were used to model equivalent aluminium deck of the bridge and MASS21 to simulate the mass of non-structural elements. Figure 5a,b show the complete bridge model and the model without the deck, respectively. Beam elements were used to model the hydraulic cylinders (Figure 5c), that together with other modelling strategies revealed essential to get right calibrated frequencies. The welded-riveted connections were assumed to be fully fixed in a finite element perspective, i.e. within the FE model this is achieved by tying all the members together at the locations of the connections fixed as shown in Figure 5d.

![Figure 5](image.png)

**Figure 5.** (a) Overview of the Global model of the bridge, (b) The model without deck, (c) Hydraulic cylinder and (d) Welded-riveted connections.

In reality, composite action between deck and cross beam is developed using shear bolts bolted to the top flange of the cross beam. In the model the composite action between the aluminum deck was modelled by connecting the top flange nodes to the deck nodes by using coupling and constraint equations technique. There were a total of 26,891 elements and 28496 nodes in the model. The quad-shaped element is used with size 150 mm. Young modulus of 205 GPa and 69 GPa reported by (Devon county council) for rolled-iron steel and aluminium consecutively, and a Poisson’s ratio of 0.3 are used for the FE analyses. The commercial finite element code ANSYS® was used to perform numerical analyses. The ANSYS® parametric design language (APDL) was used to build the model of the bridge.
4. Experimental characterization of the structure

4.1. Modal identification and numerical modal analysis

In order to update the numerical model of the bridge, a preliminary experimental modal identification was performed by two different accelerometers setups which involved an ambient vibration test and a dynamic test for the passage of a truck used in load test. Regarding the first setup, the ambient structural response was measured during period of 30 minutes at 4 points along the one of the main beam of the bridge. ARTeMIS Modal 5.1 with time domain modal analysis from accelerations data using these techniques: Stochastic Subspace Identification (SSI) with Unweighted Principal Component (SSI-UPC) was used for modal identification.

A global mode was identified from ambient vibration test that was a first global bending mode only due to a limitation with ambient test was just for one side of the bridge. The second accelerometers setup which has 4 points were distributed on both side of the bridge at quarter and mid span during period of 20 minutes. A first global bending and torsion modes were identified. Figure 7 shows stabilization diagrams associated to the application of these two accelerometers setup. The red line is “stable mode”, the other (yellow) is a “noise mode”, and both have been identified on accelerometers setups.

On the other hand, a modal analysis for the calibrated bridge model was performed using ANSYS. Only 100 modes were considered in dynamic analysis to predict the bridge performance. A first global bending and torsion modes were also identified from the numerical model. The numerical analysis has showed that both global bending and torsion modes are responsible for almost all the stress range in the main girders of the bridge. The asymmetric behaviour observed in the first bending mode is due to the difference of mass included more next to this lane due to existence of a footway. Figure 7 show a comparison between the experimental and numerical identified modes with 2% of error.

![Figure 6](image_url)

**Figure 6.** (a) Stabilization diagram of the accelerometers setup one. The red line is “stable mode” and the other (yellow) is a “noise mode” and (b) Accelerometers setup two. The red line is “stable mode” and the other (yellow) is a “noise mode”
Figure 7. (a) Comparison between 1st numerical and experimental vibration mode shape and (b) Comparison between 2nd numerical and experimental vibration mode shape.

4.2. Strain analysis

Weldable strain gauge installed within 7 cm of the exterior edge of the bottom flange as shown in Figure 8a,b and Figure 9a. To avoid any notch effect induced by the welded connection near to the weld toe, the sensor was offset a distance of 40 mm to the left of the cross beam. A controlled load test was conducted on the bridge for collecting data to calibrate the finite element models of the bridge. A truck used in the load test, with weighed axle, described in next section and shown in Figure 8. During the test the speed of the truck was measured equal to almost 17 km/h. The bridge was closed during the test for local traffic. Bending strains were measured from the live-load response at 2000 Hz sample rate. Using the FE models, the field test run using the truck was simulated. To simulate the truck pass, the two wheel loads for truck were applied as nodal forces time steps on the deck nodes at the dimensions shown in next section. At Figure b strains were multiplied by \( E = 2.05 \times 10^{11} \) to get the stress-history response in terms of Megapascals for the truck crossing. The results would indicate agreement between the FEM and the field test results, presenting a good agreement between numerical and experimental. Comparable maximum stresses would indicate agreement between the FE model and the field test results.

Figure 8. (a) Schematic of strain gauge Section and (b) Strain gauge at main beam location.
5. Fatigue life evaluation

5.1. Identification and classification of fatigue prone details

An identification and classification of bridge’s fatigue prone detail connections is presented first to calculate fatigue damage using SN method. The design information based on the Eurocode 3 [5] was used for this classification. Among the details, where are not considered in the present study, the most vulnerable position with respect to fatigue would be the fillet weld between the vertical stiffener welded to the web of the left main girder of the bridge. Figure shows the detail which is classified as class “80”. In the current paper, the fatigue damage assessment for the Bascule Bridge was limited only to this detail at an initial stage of this research work.

Figure 9: (a) SN characteristic curve Detail 80 ($\Delta \sigma_c = 80$ MPa) and (b) Potential prone fatigue joint.
5.2. Fatigue damage calculation based on design standard

In this section, an initial estimation of the fatigue damage of the aforementioned detail is presented and the research will continue studying and investigating more details on the bridge. The damage linear accumulation method, described in Eurocode 3 [5] was used to analysis the critical detail. In order to extract stress history responses of the critical detail of the bridge, a design loads based on NA to BS EN 1991-2-3 [6] were used to represent the service period from 1973 up to the present.

The most common practices for the assessment of fatigue strength of Civil Engineering structures are currently available in the Eurocodes. In general, Eurocode 3 [5] allows 3 different methods for fatigue analysis of steel structures: (a) the simplified method of equivalent constant amplitude stress range, (b) the linear damage accumulation method (Annex A) and (c) the hot-spot stress approach (Annex B). At a preliminary stage, the damage accumulation method is used in this work, and its application in the context of steel and steel-concrete roadway bridges involves the following steps Nussbaumer et al. as cited [7].

1. definition of the traffic scenario (vehicle types, annual traffic volume);
2. calculation of the stress history in potential critical structural details, including the dynamic effects on structural response;
3. calculation of stress histograms, representing the number of cycles versus the corresponding distribution of stress ranges, by using a cycle counting algorithm, e.g. the rainflow algorithm [8];
4. adoption of suitable SN curves to describe the fatigue resistance of the detail; the curve proposed by the Eurocode 3 [5] are identified; moreover, as stated by NA BS EN 1993-1-9 [4], the fatigue strength reduction factor, \( \gamma_{M6} \), should be taken as 1.1;

5. computation of the fatigue damage, by using a linear damage accumulation model, as proposed by Miner[8], according to which the damage factor, \( D \), is given by Equation (1), where \( n_i \) is the number of applied load cycles for a given stress range and \( N_i \) denotes the number of resisting load cycles for a given stress range. Fatigue failure is reached for \( D = 1 \).

\[
D = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \ldots = \sum_{i=1}^{k} \frac{n_i}{N_i} \leq 1.0
\]

5.2.1. Stress ranges calculation

Although the estimation of the fatigue damage obtained based Eurocode 3 [5], there are assumptions adopted based on the UK specific National Annex NA BS EN 1991-2-3 [6]. The bridge was loaded in both lanes with the Fatigue Load Model 4 Lorries as described in NA BS EN 1991-2-3 [6] and shown in Table 1. These lanes are the two notional lanes that individually cause the most theoretical fatigue damage in the component under consideration. Vehicle numbers in these lanes were obtained from Table NA4 in NA BS EN 1991-2-3 [6]. An equal annual frequency of the vehicles was assumed to represent the period between 1973, opening time, to the 2017. Fatigue damaging stress cycles in the investigated detail were calculated from two traffic lanes based on NA BS EN 1991-2-3 [6] and counted using the rainflow algorithm [9]. The calibrated FE model of the bridge was used to extract the fatigue stress time history.

| Chassis type | Average Spacing (m) | Loading Group | Total Weight kN | Axle Loads kN |
|--------------|---------------------|---------------|----------------|--------------|
| Girder       |                     | H             | 3680           | 80 160       |
| trailer and  |                     | M             | 1520           | 80 160 160  |
| 2 tractors   | 1.5 4.5 4.5 [No.5] | H             | 1610           | 70 140 140   |
|               | 4.5 1.5 4.0 1.5     | M             | 750            | 50 110 110   |
| Girder       |                     | H             | 1310           | 70 140 140   |
| trailer and  |                     | M             | 680            | 60 130 130   |
| 2 tractors   | 4.5 1.5 4.0 2.0     | H             | 1310           | 70 140 140   |
| Articulated  | 3.0 1.5 9.5         | H2            | 630            | 70 130 130   |
| Articulated  | 3.0 1.5 9.5         | H             | 380            | 70 100 70    |
| Articulated  | 3.0 6.5             | M             | 300            | 50 70 60     |
| Rigid        | 1.5 3.5             | L             | 145            | 35 50 30     |
| Rigid        | 1.5 4.0             | M             | 240            | 40 80 60     |
| Rigid        | 4.0                 | L             | 120            | 20 20 20     |

Table 1. Set of equivalent lorries for Fatigue Load Model 4 NA to BS EN 1991-2-3 [6].
Figure 11 illustrates the different stress history responses of the investigated detail of the bridge to the passage of the 10 heaviest standard fatigue lorries. Dynamic analyses using the Fatigue Vehicles with a crowd speed of 17 km/h were performed on the basis of the dynamic superposition method using ANSYS and Matlab, after a parametric analysis varying speed between 10 and 120 km/h had shown relatively low influence over the stress ranges. In this work, dynamic analyses were performed using a damping ratio value of 2.45% for all vibration modes, due to the fact that this was the damping ratio measured from the ambient test for the 1st vibration mode.

It can be seen that the behaviour was axle-dominated and that the stresses vary between 6.5 and 86 MPa. In this case the damage was vary and the most conditioning vehicle was the vehicle 15. Total damage summation, $D_d$, is obtained by adding contributions from the Lane 1 and Lane 2 traffic as given in Equation (2).

$$D_d = D_1 + D_2$$
where $D_1, D_2$ ate the damages from Lane 1 and Lane 2 respectively. The effect of side-by-side running also was allowed as recommended in NA to BS EN 1991-2-3 [6] and given by Equation (3).

$$D_a = K_b \cdot Z$$

where: $K_b =$ ratio of the maximum stress range caused by single vehicles in lane 2 to the maximum stress range caused by single vehicles in Lane 1, and $Z$ varies linearly in proportion to the logarithm of the loaded length from 1.0 to 1.5 for loaded lengths between 3.0 m and 20.0 m. In this case study the $K_b$ and $Z$ were calculated as 1 and 1.448, respectively. Figure 12 shows the total damage for each fatigue vehicle types that were considered in this study and the total annual damage for all vehicles was $2.428 \times 10^4$, considering 1 million vehicles per year per lane.

However, this damage was for 2 million vehicles (1 million per lane per day). As the bridge is located at the A379 roadway, classified as all-purpose type, has a single traffic categories and two lanes only. From Table NA4 BS NA EN 1991-2 [6] number of heavy goods vehicles expected per year was used 0.5 million per lane. Therefore, the total annual damage become $1.214 \times 10^4$. Finally, the total damage for the critical for a 42-year period is $5.1 \times 10^3$.

## 6. Conclusions

The steel Bascule Exeter Bridge is under study concerning its fatigue behaviour. In this kind of analysis, the finite element model of the structure is a very important tool. In the present work a finite element mode of the bridge was calibrated through measured responses for ambient vibration test and updated using output-only modal techniques. Strain measurement recorded from dynamic testing of the bridge was used to crosscheck results. A good agreement between experimental strains and numerical were achieved, this was possible due to a calibration of the first two global modes with less than 2%, that for strains measured in the main girders were responsible for great part of the numerical response computed using the dynamic mode superposition method. Analysis of the fatigue damage was conducted using code-specified loadings. Among all the stress range from the Fatigue Vehicles the higher stress range is obtained for the Standard Fatigue Vehicle 23. Moreover, in this case study, the damage in the structural detail 80 was vary and the most conditioning the Vehicle 15. In general, low damage values were found.
This study is currently under way in order to further validate these results and obtain better estimates of fatigue life. Future work will also focus on assessing other potential fatigue prone details combining both methodologies to update and calibrate a model: (1) static analysis versus strain measurements from static load test (2) dynamic analysis (modal and transient) versus acceleration measurements. In addition, study using traffic survey is under way to predict fatigue damage at fatigue prone details of the bridge. Finally, a local model, which will allow better assessment of local stresses, is currently being developed for investigating model uncertainties such as a mesh sensitivity and lifting rods.

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