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REFINED FATIGUE ASSESSMENT OF AN EXISTING STEEL BRIDGE

John Leander, Raid Karoumi

Abstract: This paper treats the fatigue assessment of existing steel bridges for road traffic. It is focused on the estimation of the load effect. The deterministic assessment methods suggested in governing codes are reviewed and a comparison is performed against a reliability-based assessment. The latter method enables a consideration of reduced uncertainties from measurements of the real load effect. The Vårby Bridge in Sweden, a steel–concrete composite bridge south of Stockholm, is used as a case study. The results show a considerable increase in fatigue life with the use of measurements and a reliability-based assessment. Another conclusion is that the load models in the Eurocode give an unjustified conservative result.

1 Introduction

The service life for steel bridges is typically limited by fatigue. Environmental effects as corrosion and wear can contribute to the deterioration and a decreased fatigue resistance. The uncertainties in these circumstances, the fatigue resistance itself, and the actual loads on bridges, entail a need for many conservative assumptions in design. In the codes for design of new bridges this is reflected by characteristic values of the fatigue resistance determined for fatigue tests with a large scatter and, moreover, conservative load models. This approach is indeed justified in design. However, in assessment of existing bridges more accurate methods should be recommended. For economic and environmental reasons, effort should be put on keeping existing structures in service for as long as possible. Actions as strengthening or replacement must be based on reliable predictions.

The conventional assessment methods following the Eurocodes EN 1993-2 [1] and EN 1993-1-9 [2] are reviewed. The basic features of the methods are summarized including the resistance and the load. The load is in this case considered with standardized vehicles with prescribed axle loads and a prescribed number of passages. The actual load is one of the main uncertainties since a road bridge is loaded with a large variety of vehicles from light cars to heavy lorries.

To reduce the uncertainties in the load effect, Helmerich et al. [3] suggests in situ measurements. By long term measurements, a representative stress range spectrum for the instrumented detail can be recorded and used in the fatigue assessment. The stress history is typically determined by strain measurements using electrical strain gauges [4]. The Eurocode EN1993-2 supports the use of recorded traffic data as a load model, however, no guidance is given for the use of in situ strain measurements. In this paper a reliability-based model presented in Leander et al. [5] to incorporate measured stresses is used. It enables an assessment against a
target reliability and a consideration of uncertainties related to the model and the measured response.

As a case study, the Vårby Bridge south of Stockholm in Sweden is used. It is a continuous steel–concrete composite bridge carrying the traffic on the highway E4. It was completed in 1996 and designed according to the contemporary Swedish codes, before the Eurocodes were taken in action. Fig. 1 shows a photo of the bridge.

Fig. 1: The Vårby Bridge south of Stockholm, Sweden. Photo: &Rundquist (www.rundquist.se).

2 Assessment based on theoretical load models

Following the Eurocode EN 1993-2 [1], a fatigue assessment should be performed for fatigue load model 3 or 4. The former is a single vehicle model and the latter is a set of standard lorries with a defined distribution. The different vehicles are described in EN 1991-2 [6]. With allowance from the owner, fatigue load model 5, based on recorded traffic can be used [7].

2.1 Fatigue load model 3 (FLM3)

The single vehicle of fatigue load model 3 (FLM3) consists of four axles with the weight of 120 kN each. The geometry is shown in Fig. 2.

Fig. 2: Fatigue load model 3, a single vehicle model. Axle distances are given in meters.

From the response of the single vehicle, the real stress range spectrum is estimated by the use of so-called damage equivalence factors, also called lambda factors. A damage equivalent stress range related to 2 million cycles is calculated as [1]

\[ \Delta \sigma_{p2} = \lambda \Phi_2 \Delta \sigma_p \]  

where \( \lambda \) is the damage equivalence factor, \( \Phi_2 \) is the damage equivalent impact factor, and \( \Delta \sigma_p \) is the reference stress range calculated for the load model. The fatigue resistance is verified as
where $M_{FLM3}$ is as safety margin, $\gamma_{Ff}$ and $\gamma_{Mf}$ are partial safety factors and $\Delta \sigma_C$ is the fatigue strength of the studied detail. The fatigue strength is typically described by bilinear $S–N$ curves for axial stress and linear curves for shear stress, see EN 1993-1-9 [2].

The lambda factor, $\lambda$, mentioned above should be calculated as a product of four different factors as

$$\lambda = \lambda_1 \lambda_2 \lambda_3 \lambda_4 \leq \lambda_{\text{max}}$$

where $\lambda_1$ to $\lambda_4$ consider the influence length, the traffic volume, the design life and the number of lanes, respectively. The final value is limited to $\lambda_{\text{max}}$ taking account of the fatigue limit [1]. The properties of the traffic is covered by the load model itself and the factor $\lambda_2$. The combination of these values should reflect the actual traffic situation. A weighing formula for $\lambda_2$ is given in [1] as

$$\lambda_2 = \frac{Q_{m1}}{Q_0} \left( \frac{N_{\text{obs}}}{N_0} \right)^{1/5}$$

where $Q_{m1}$ is the average gross weight of the lorries in the slow lane, $Q_0$ is the total weight of the load model, $N_{\text{obs}}$ is the total number of heavy vehicles per year in the slow lane, and $N_0$ is a constant equal to $0.5 \times 10^6$. Through Eq. (4), a relation is given between the load model and the actual statistics of the lorries travelling at the site.

### 2.2 Fatigue load model 4 (FLM4)

The next level of complexity, after FLM3, is to consider a more realistic traffic mix. A set of standard lorries are given in EN 1991-2 [6] which together should produce effects equivalent to those of typical traffic. The configuration of these lorries is shown in Fig. 3.

![Fatigue load model 4](image)

**Fig. 3:** Fatigue load model 4, a set of standard lorries. Axle distances are given in meters.

The stress history should be calculated for all five lorries. The distribution of lorries for different traffic types is given in EN 1993-2. The total number of lorries per year is determined by $N_{\text{obs}}$ mentioned in relation to FLM3. A stress range spectrum should be derived using some
cycle counting technique such as the rainflow method or the reservoir method. Descriptions of these methods can be found in, e.g., the textbook by Stephens et al. [8].

The fatigue verification is performed for the derived stress range spectrum using the Palmgren–Miner rule [9, 10]. The criterion is stated in EN 1993-1-9 as

$$D = \sum \frac{n_{Ei}}{N_{Ri}} \leq 1 \quad (5)$$

where $n_{Ei}$ is the number of cycles with the stress range $\Delta\sigma_i$ and $N_{Ri}$ is the number of cycles to failure for the same stress range. By inserting the expressions for the fatigue endurance and partial safety factors a safety margin can be expressed as

$$M_{FLM,4} = D - \frac{\left(\gamma_{FL} \gamma_{ML}\right)^{m_1}}{2 \times 10^6 \Delta\sigma_C^{m_1}} \sum_i n_{Ei} \Delta\sigma_i^{m_1} - \frac{\left(\gamma_{FL} \gamma_{ML}\right)^{m_2}}{5 \times 10^6 \Delta\sigma_D^{m_2}} \sum_j n_{Ej} \Delta\sigma_j^{m_2} \quad (6)$$

which considers the bilinear shape of the $S$–$N$ curve in EN 1993-1-9. The stress ranges with index $i$ and $j$ are the stress ranges above and below the knee point $\Delta\sigma_D$, respectively. The variable $m$ in Eq. (6) is the slope of the $S$–$N$ curve with values $m_1 = 3$ and $m_2 = 5$. The knee point is related to the fatigue strength as

$$\Delta\sigma_D = \left(\frac{2}{5}\right)^{1/3} \Delta\sigma_C \quad (7)$$

3 Assessment based on measured response

The Eurocode does not expressly support the use of in situ measurements. Fatigue load model 5 mentioned earlier should be based on recorded traffic data, not on measured strains. However, the Swedish code [11] for assessment of existing bridges allows the use of measured response.

An assessment based on load models relies on a theoretical structural analysis. By measuring strains close to a fatigue critical detail the stress range spectrum from the real traffic can be recorded. Questions to be dealt with are the positioning of the gauges, the duration of measurements and the quality of the response. These topics have been treated in, e.g., [5], [12] and [13]. If nominal stresses are to be used in the assessment the gauges have to be located away from local stress raisers. Adequately positioned, the measured strains can be used directly in the fatigue assessment. Measured strain is converted to stresses by Hookes’ law and a stress range spectrum can be derived by rainflow analysis [14]. The verification is typically performed based on the accumulated damage as for fatigue load model 4 with Eq. (6).

4 Reliability-based assessment

The inherent uncertainties in the loads, the response and the resistance can be modelled explicitly by a reliability-based assessment.

4.1 Limit state equation

Starting with Eq. (6) a limit state equation can be formulated as [5]

$$g = \delta - \frac{X_S^{m_1}}{K_1} \sum_i n_{Ei} \Delta\sigma_i^{m_1} - \frac{X_S^{m_2}}{K_2} \sum_j n_{Ej} \Delta\sigma_j^{m_2} \quad (8)$$
where $\delta$ is the stochastic variable related to the accumulated damage and considers the uncertainty of the Palmgren–Miner rule. The variable $X_S$ represents the uncertainties in the measured response. $K_1$ and $K_2$ are stochastic variable related to the fatigue endurance. A state of failure is defined for $g \leq 0$ and the probability of failure is defined as $P_f = P(g \leq 0)$ [15]. The reliability index $\beta$ is related to the probability of failure as

$$\beta = -\Phi^{-1}(P_f)$$

where $\Phi^{-1}(\cdot)$ is the inverse of the standardized normal distribution function. The results presented in this paper have been calculated using the first order reliability method (FORM) and an iteration scheme described in Madsen et al. [16]. The stochastic variables are described by their distribution functions, mean values, and standard deviations. Nonnormal distributed variables have been considered using the normal tail approximation [15, 16]. The stochastic variables are assumed to be independent except for $K_1$ and $K_2$ which are considered as fully correlated.

### 4.2 Uncertainties

The model uncertainty of the Palmgren–Miner rule is considered by the variable $\delta$ in Eq. (8). In the JCSS Probabilistic model code [17] this variable is suggested to have a lognormal distribution with a mean value of unity and a coefficient of variation (CoV) of 0.3. The origin is attributed to Wirsching [18].

The uncertainty of the measured stress ranges is considered by the variable $X_S$. It should include the accuracy of the strain gauges and other devices of the monitoring system. I should also include the effect of misalignment in the positioning of the gauges. In Frangopol et al. [19], the measurement uncertainty is suggested to have a lognormal distribution with a unit mean and a CoV of 4%.

The fatigue endurance depends on the stress range and the characteristics of the $S–N$ curve. In the model described by Eq. (8) the stress range $\Delta \sigma$ and the slope $m$ are considered as deterministic. The stochastic nature of the fatigue endurance is considered by the variables $K_1$ and $K_2$. The mean value of $K_1$ is determined based on the characteristic value of the fatigue strength as

$$\mu_{\ln K_1} = \ln K_C + k s_{\ln K}$$

where $K_C$ is the characteristic value of $K_1$, $k$ is a tolerance interval factor set to $k = 2$, and $s_{\ln K}$ is the standard deviation for $K_1$. With a known or assumed CoV from fatigue tests, the standard deviation can be calculated as

$$s_{\ln K} = \sqrt{\ln(V_K^2 + 1)}$$

where $V_K$ is the CoV for the fatigue endurance. For this study, a fixed value of $V_K = 0.49$ is used following the motivation in [5]. The knee point of the bilinear $S–N$ curves in the Eurocode is fixed at 5 million cycles. Thereof, $K_2$ can be formulated fully correlated to $K_1$ as

$$K_2 = \left[ \frac{K_1^5}{(5 \times 10^6)^{\frac{1}{3}}} \right]^{1/3}$$

which is valid for $m_1 = 3$ and $m_2 = 5$. The stochastic variables used in this paper are summarized in Table 1.
Table 1: Stochastic variables. N ~ Normal, LN ~ Lognormal, DET ~ Deterministic.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution</th>
<th>Mean</th>
<th>CoV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta$</td>
<td>LN</td>
<td>1</td>
<td>0.3</td>
</tr>
<tr>
<td>$X_S$</td>
<td>LN</td>
<td>1</td>
<td>0.04</td>
</tr>
<tr>
<td>$\ln K_1$</td>
<td>N</td>
<td>$\mu_{\ln K}$</td>
<td>0.49</td>
</tr>
<tr>
<td>$m_1$</td>
<td>DET</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>$m_2$</td>
<td>DET</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>$K_2$</td>
<td>Fully correlated to $K_1$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5 The Vårby Bridge

The Vårby Bridge south of Stockholm in Sweden is used as a case study. It is composed by two parallel steel–concrete composite bridges carrying the highway E4 between Stockholm and Södertälje. Both bridges have the same design and are continuous bridges in six spans with the total length of 255 meters. The span lengths are 38 m and 44 m for the end spans and the intermediate spans, respectively. Each cross-section is built up by two I shaped steel beams and a concrete deck. Only the north bridge will be treated henceforth. A schematic view of the cross-section is shown in Fig. 4.

The bridge was completed in 1996 and designed in accordance with the contemporary Swedish codes. The Eurocodes were not in force at that time. About ten years after its completion, fatigue cracks were found during routine inspections in the joint between the vertical web stiffeners and the top flange of the steel beams. Possible reasons for these cracks are discussed in the licentiate thesis by Nilsson [20]. The cracks are not the subject of this paper, however, the disclosure initiated a monitoring campaign which result have been used in this study. A brief description of the monitoring campaign and a summary of the results can be found in [21].

For Swedish conditions the traffic volume on the bridge is high. The annual mean day traffic is about 40 to 50 thousand vehicles in one direction. Heavy vehicles with a total weight of 3.5 metric tonnes or more constitute about 10% of the total volume.

The material of the main beams is of steel grade 2134. The concrete in the deck is of grade K40. Full composite action between the steel beams and the concrete deck is assured by shear studs. The steel beams have a varying height as can be seen in Fig. 1 which means that the stiffness is varying along the bridge. Another cause of a varying stiffness is the occurrence of cracks in the concrete. The deck is assumed to be cracked over the supports.
To clarify the real behaviour of the bridge, calibration measurements have been performed. The response was recorded for several passages of a lorry with known axle distances and weights. The properties of the lorry are shown in Fig. 5(a).

Measured strains recalculated to stresses for two passages are shown in Fig. 5(b). They are measured at the section of a cross girder in the second span. The gauge is located at the centre of the bottom flange of the most utilized beam. See gauge 8 in Fig. 4 and the description of the monitoring campaign in [20]. Fig. 5(b) shows also a theoretically calculated stress history for the calibration lorry. The theoretical model is a simple two dimensional beam model created in the FE program Lusas. The FE program is used to create an influence line only. The traffic evaluation is performed in Matlab using the principle of superpositioning. A simplified updating of the theoretical model is performed by scaling the influence line to reach the same maximum stress range as recorded by the calibration measurements.

![Diagram](image)

(a) Vehicle. (b) Theoretical and measured response.

**Fig. 5:** The vehicle and the response for the calibration. P1 and P7 stand for passage 1 and passage 7, respectively. The response is valid for the location of strain gauge 8.

The oscillation in the measured response shown in Fig. 5(b) indicates a dynamic behaviour. This is not captured in the static theoretical analysis. However, the oscillations are small in comparison to the maximum stress range and will not cause any stress ranges above the fatigue limit. The theoretical model is updated against the maximum stress range and, thereby, contains the influence of dynamic amplification.

### 6 Results

The results presented are calculated for the instrumented section of the Vårby Bridge. The fatigue assessments are performed as parametric studies and should not be confused with the fatigue capacity of the real bridge. Possible fatigue prone details are shown in Fig. 6 together with their S–N curves according to EN 1993-1-9. Detail (a) corresponds to a crack initiated in the butt weld between the web and the flange. Detail (b) corresponds to a crack initiated at the weld toe of a transverse stiffener. Detail (c) corresponds to a crack initiated at an in-plane gusset plate without any transition radius.

#### 6.1 Theoretical load models

The assessment based on theoretical load models requires an estimated number of heavy vehicles per year in the slow lane, \( N_{\text{obs}} \) in Eq. (4). Based on an annual mean day traffic of more than 6000 heavy vehicles (in both directions), the bridge should be verified for traffic category 1 [22]. This leads to \( N_{\text{obs}} = 2 \times 10^6 \) heavy vehicles per year according to EN 1993-2. That is a quite extreme traffic situation which is representative only for a few number of bridges. For this study, traffic category 2 have been used which gives \( N_{\text{obs}} = 0.5 \times 10^6 \).
The result for FLM3 is shown as the safety margin defined in Eq. (2) for which $M_{FLM3} \geq 0$ indicates a sufficient capacity. Following EN 1993-2 the lambda factors are determined to $\lambda_1 = 2.21$, $\lambda_2 = 0.85$, and $\lambda_4 = 1$. The factor $\lambda_3$ varies from 0.72 to 1.34 for a service life of 20 to 200 years. The resulting lambda value will reach the maximum value of $\lambda \leq \lambda_{\text{max}} = 2$ after 137 years. The time history for the passage of FLM3 is calculated using the updated theoretical beam model and shown in Fig. 7(a). The reference stress range is $\Delta \sigma_p = 36.2$ MPa. Partial safety factors $\gamma_{Ff} = 1$ and $\gamma_{Mf} = 1.35$ gives the safety margin shown in Fig. 7(b). For a detail category of 40, the fatigue life is consumed already after 20 years indicated by a negative safety margin. For detail category 80 the fatigue life is estimated to about 50 years. An infinite fatigue life is reached for a detail category of 125 which is an effect of the fatigue limit. The plateaus for $T > 137$ are due to the condition $\lambda_{\text{max}}$ which is related to the fatigue limit.

A verification using fatigue load model 4 is also dependent on $N_{\text{obs}}$, the number of heavy vehicles per year in the slow lane. A traffic type classified as medium distance is used which gives the percentages 40, 10, 30, 15, and 5 for lorries (a) to (e) in Fig. 3 [6]. The complete stress range spectrum for all lorries is shown in Fig. 8(a). It is derived using the rainflow cy-
cle counting method. Every lorry gives a stress history similar to the one for FLM3 shown in Fig. 7(a).

The safety margin for fatigue load model 4 is calculated using Eq. (6) based on the accumulated damage and shown in Fig. 8(b). The partial safety factors are set to $\gamma_{Ff} = 1$ and $\gamma_{Mf} = 1.35$ as before. A fatigue limit is considered defined as [2]

$$\Delta\sigma_L = \left( \frac{5}{100} \right)^{1/5} \Delta\sigma_D$$

(13)

where $\Delta\sigma_D$ is defined by Eq. (7). All stress ranges below the fatigue limit are omitted in the damage accumulation. As for FLM3, Fig. 8(b) shows an infinite fatigue life for a detail category 125, a finite life for category 80, and an exhausted life already after 20 years for category 40. For detail category 80 the fatigue life is estimated to about 67 years.

6.2 Measured stresses

The measured stresses were recorded at the end of June and beginning of July 2009. The total duration corresponds to about three days. The short duration of the measurements makes the estimation of a representative traffic volume highly uncertain. In lack of other reliable data for the present traffic volume, the three days of measurements have been extrapolated linearly to estimate the yearly traffic. Two stress range spectra from the measurements are shown in Fig. 9(a). Gauge 8, located on the beam carrying the larger portion of the load in the slow lane, has the greatest number of cycles for all stress ranges. This is a logical outcome since most lorries are expected to run in the slow lane. The subsequent result is therefore based on the spectrum for gauge 8.

A deterministic verification for the measured response can be performed using the same equation for the safety margin as for FLM4 specified in Eq. (6). The deterministic safety margin for the stress range spectrum of gauge 8 is shown in Fig. 9(b). It is calculated with partial safety factors $\gamma_{Ff} = 1$ and $\gamma_{Mf} = 1.35$ and the consideration of a fatigue limit. For detail category 125 the result is an infinite fatigue life as before. The safety margin for category 80 is declining with time but shows sufficient capacity for at least 200 years. The fatigue life is exhausted already after 20 years for a detail category of 40 but not as excessively as for FLM3 and FLM4.
The deterministic fatigue assessment implies the same uncertainty in the measured response as for a theoretically calculated response. To utilize the reduction of uncertainties, which the measurements enable, a reliability-based assessment is performed. The reliability index $\beta$ is calculated for the limit state equation, Eq. (8), using the FORM and the stochastic variables stated in Table 1. The result is shown in Fig. 10 as the reliability index over time. Two reliability levels are indicated in Fig. 10, $\beta = 3.1$ and $\beta = 2.3$. These are the target reliability indices stated in ISO 13822 [23] for fatigue assessment of existing structures. The higher value is suggested for non-inspectable components and the lower value for inspectable components. For detail categories 125 and 80 the reliability is higher than the target $\beta = 3.1$ for a service life of more than 200 years. For detail category 40, the target reliabilities $\beta = 3.1$ and $\beta = 2.3$ are reached after 24 and 41 years, respectively.

**Fig. 10:** Reliability index for the measured response.

### 6.3 Summary of results

The results for the different assessment methods are summarized in Table 2.

**Table 2:** Results of the different assessment methods.

<table>
<thead>
<tr>
<th>Method</th>
<th>Traffic</th>
<th>$\Delta\sigma_C = 125$ MPa</th>
<th>$\Delta\sigma_C = 80$ MPa</th>
<th>$\Delta\sigma_C = 40$ MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deterministic</td>
<td>FLM3</td>
<td>$\infty$</td>
<td>50</td>
<td>$&lt; 20$</td>
</tr>
<tr>
<td></td>
<td>FLM4</td>
<td>$\infty$</td>
<td>67</td>
<td>$&lt; 20$</td>
</tr>
<tr>
<td></td>
<td>Real</td>
<td>$\infty$</td>
<td>$&gt; 200$</td>
<td>$&lt; 20$</td>
</tr>
<tr>
<td>FORM, $\beta = 3.1$</td>
<td>Real</td>
<td>$&gt; 200$</td>
<td>$&gt; 200$</td>
<td>24</td>
</tr>
<tr>
<td>FORM, $\beta = 2.3$</td>
<td>Real</td>
<td>$&gt; 200$</td>
<td>$&gt; 200$</td>
<td>41</td>
</tr>
</tbody>
</table>
A component with a detail category as high as 125 is in this case insensitive to the analysis method used. The fatigue limit for this category is high enough to dismiss most occurring stress ranges. As seen in Table 2, the fatigue life is for all methods longer than 200 years.

For a detail category of 80, a significant increase in fatigue life is reached when measured stresses are used instead of the theoretical load models. With measurements, a fatigue life longer than 200 years is reached. With the load models in the Eurocode, the bridge would not have sufficient resistance to withstand a design life of 120 years.

The stresses in the studied section are too high to allow a component with detail category 40. This is shown in Table 2 as a fatigue life shorter than 20 years for all deterministic verifications. The measured response and the reliability-based assessment do, however, allow a significant increase in the fatigue life. Without inspections a fatigue life of 24 years is reached. With inspections and the acceptance of a lower reliability level, a fatigue life of 41 years is reached.

7 Conclusions

The following conclusions are based on the fatigue assessment of a specific section of the Vårby Bridge in Stockholm, Sweden. The assessment is performed with different methods and for three different connection details frequently occurring in steel bridges.

1. The load models from the Eurocode gives a conservative estimate of the fatigue life in comparison to the life determined for the measured response. For detail category 80, the fatigue life is estimated to 50 and 67 years for FLM3 and FLM4, respectively. A deterministic assessment based on measured response from real traffic gives a fatigue life longer than 200 years.

2. A reliability-based assessment using measured response increases the estimated fatigue life even further.

3. The reliability-based assessment shows that the partial safety factors used in the deterministic verification are appropriate also for measured response. A comparison of Fig. 9(b) and Fig. 10 shows that the fatigue life is estimated to about 20 years for a detail category of 40 for both methods.

4. For this specific bridge and the studied section, the fatigue load model 3 in the Eurocode limits the service life to 50 years for detail category 80. The reliability-based assessment gives a reliability index of about $\beta = 6$ for the same service life which is significantly higher than the suggested target reliabilities in ISO 13822.

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