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FATIGUE RELIABILITY OF AGEING RAILWAY BRIDGES: FEASIBILITY OF PROBABILISTIC APPROACH

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Abstract. *Miner's rule employed deterministic or probabilistic fatigue assessment approaches are generally used to predict remaining fatigue life of ageing railway bridges. Under many variable amplitude loading conditions, life predictions have been found to be unreliable since Miner's rule does not properly take account the loading sequence effect. Therefore, this paper presents a comparison of a new probabilistic fatigue assessment approach with deterministic approach consisting of a new damage indicator, which captures the loading sequence effect of variable amplitude loads more precisely than the Miner's rule. The comparison is performed by applying both fatigue assessment approaches to predict the remaining fatigue life of an ageing railway bridge. Initially the paper presents a probabilistic stress-life approach. Then the proposed approach is applied to predict the remaining fatigue lives of the ageing railway bridge. Finally, predicted fatigue lives are compared with fatigue lives predicted by the deterministic approach. Hence applicability, significance and validity of the proposed approach is discussed.*

1 INTRODUCTION

Majority of the railway bridges in the world are exceeding their design lives and bridge authorities are working on precious life extension methods [1-3]. As a result, a significant amount of research are ongoing for development of precious structural health monitoring and life assessment methods [2-12]. As railway bridges are vulnerable for time-dependent fatigue damage due to cyclic nature of traffic loads, the assessment of remaining fatigue life of railway bridges for continuing services has become more important than ever, especially when making decisions regarding structure replacement and other major retrofits. However, this task is difficult due to the increase of axel load and corrosion deterioration on bridges.

The fatigue assessment of structures mainly done by either deterministic or probabilistic approach. Most of the present day deterministic fatigue assessment approaches of railway bridges are generally based on the combination of measured stress histories under actual traffic load [12,13], Miner's rule [14] and railway code provided fatigue curve (also referred S-N or Wöhler curve). Although the mentioned deterministic approach predicts the remaining fatigue life, the uncertainties inherent in the fatigue evaluation process are not captured. These uncertainties are found in the process of determination of stress histories (i.e. structural analysis, field measurements, load testing, loading sequence and respective histories), selecting detail categories, choosing fatigue damage theories [15, 16].

The probabilistic fatigue assessments have been originated to capture the effect of these uncertainties more precisely. This approach is generally based on probability of fatigue failure associated reliability index. Fatigue reliability index provides a tool for predicting the remaining fatigue life [16]. A number of studies on the reliability analysis have been done for fatigue life prediction of bridges. Imam et al. [17] proposed a probabilistic fatigue assessment methodology for riveted railway bridges under historical and present-day train loading. Kwon and Frangopol [18] performed fatigue reliability assessment of steel bridges using the probability density function (PDF) of the equivalent stress ranges obtained by field measurement data. Ni et al. [19] proposed a fatigue reliability model for fatigue life and reliability evaluation of steel bridges with long-term monitoring data, which integrates the probability distribution of hot spot stress range with a continuous probabilistic formulation of Miner's damage cumulative rule. Recently, Kwon et al. [15] and Soliman et al. [16] proposed a probabilistic bilinear stress-life approach for better fatigue assessment of steel bridges. Miner's rule has been used as the fatigue damage theory for mentioned probabilistic models.

The Miner's rule is the simplest and the most commonly used fatigue life prediction technique. One of its interesting features is that life calculation is simple and reliable when the detailed loading history is unknown. However, under many variable amplitude loading conditions, life predictions have been found to be unreliable since it does not properly take account the loading sequence effect [20-22]. Therefore, it is uncertain to use the Miner's rule for remaining fatigue life estimation of railway bridges because most of the railway bridges are subjected to variable amplitude loadings. None of research studies confirmed about the consideration of the loading sequence effect on probabilistic fatigue assessment approaches.

To overcome this problem to some extent, objective of this paper is to compare probabilistic fatigue assessment approach with deterministic approach consisting of a new damage indicator (i.e. damage stress model) [22], which captures the loading sequence effect of variable amplitude loads more precisely than Miner's rule. The comparison is performed by applying both fatigue assessment approaches to predict the remaining fatigue life of an ageing railway bridge. This comparison provides an indication of feasibility of probabilistic stress-life fatigue approach for ageing railway bridges.

2 FATIGUE RELIABILITY INDEX

This section proposes a method to determine fatigue reliability index of bridges based on probabilistic bilinear *S-N* approach. The reliability index provides a measure of fatigue damage of the considered detail category of the bridge. Reliability index defines as the probability of violating fatigue limit state. The fatigue reliability index is defined as,

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

where $\emptyset^{-1}$ is the inverse the standard normal distribution function. The corresponding fatigue limit state function can be derived as,

$$g(t) = \Delta - D \tag{2}$$

where Δ is Miner's critical damage accumulation index, which is assumed to be lognormal distribution with a mean value of 1.0 and coefficient of variation (COV) of 0.3 and *D* is the Miner's damage accumulation index, which can be derived as,

$$D = \begin{cases} \frac{N(t)}{A_1}(S\,^L_{re})^{m_1} & for\ N(t) \leq \frac{A_1}{CAFT^{m_1}} \\ \frac{N(t)}{(CAFT^{m_2-m_1}.A_1)}(S\,^B_{re})^{m_2} & for\ N(t) > \frac{A_1}{CAFT^{m_1}} \end{cases} \tag{3}$$

where $S\,^L_{re}$ and $S\,^B_{re}$ are equivalent constant amplitude stress ranges calculated using linear and bilinear *S-N* approach respectively as shown in Eq (4). The *CAFT* designated as constant amplitude fatigue threshold. The m_1 and m_2 are slopes of stress-life fatigue curve above and below the *CAFT*, respectively. The A_1 is the fatigue detail coefficient above the *CAFT* of the fatigue curve. The $A_2 = A_1 CAFT^{m_2-m_1}$, which is the fatigue detail coefficient bellow the *CAFT* of the fatigue curve. The *N(t)* is the number of cycles that considered detail category has subjected at the life time of *t*. The m_1, m_2, *CAFT* and *N(t)* are considered as the deterministic parameters. The stress range S_{re}, fatigue detail coefficient A_1 are considered as random variables.

As bridges are generally subjected to variable amplitude stress cycles, the equivalent constant amplitude stress range S_{re} can be calculated for bilinear *S-N* approach as [16],

$$S_{re} = \left[\frac{\sum(n_i^o S_{ri}^{m_1}) + (CAFT^{m_1-m_2}).\sum(n_j^o S_{rj}^{m_2})}{\sum(n_i^o) + \sum(n_j^o)}\right]^{1/m_1} \tag{4}$$

where n_i^o is the number of cycles in stress range bin S_{ri} greater than the *CAFT* and n_j^o is the number of cycles in stress range bin S_{rj} less than the *CAFT*. The $\sum(n_i^o) + \sum(n_j^o)$ is the total number of cycles. Alternatively, the equivalent constant amplitude stress range S_{re} can be calculated using the PDF of the stress ranges as follows [16],

$$S_{re} = \left[\int_0^{CAFT}(CAFT^{m_1-m_2}).S^{m_2}.f_s(s).ds + \int_{CAFT}^{\infty} S^{m_1}.f_s(s).ds\right]^{1/m_1} \tag{5}$$

The Eqs. (2) and (3) can be used to calculate fatigue reliability index (β) by using Monte Carlo simulation employed some of the softwares *R*, *RELSYS*, *CALREL* or etc. The fatigue reliability index (β) versus lifetime of bridge should be plotted and compared with target reliability index (β_{target}) to determine the fatigue life of each detail category.

3 CONSIDERED RAILWAY BRIDGE

The selected bridge is one of the longest railway bridges in Sri Lanka spanning 160m (Fig.1). It is a six span-riveted bridge with double lane rail tracks having warren type semi through trusses supported on cylindrical piers. The bridge deck is made of wrought iron and the piers are made of cast iron casings with infilled concrete. The bridge was constructed in 1885 and is located in marine environment. The bridge components have been categorized to several groups

Fig.1 General views of the considered bridge

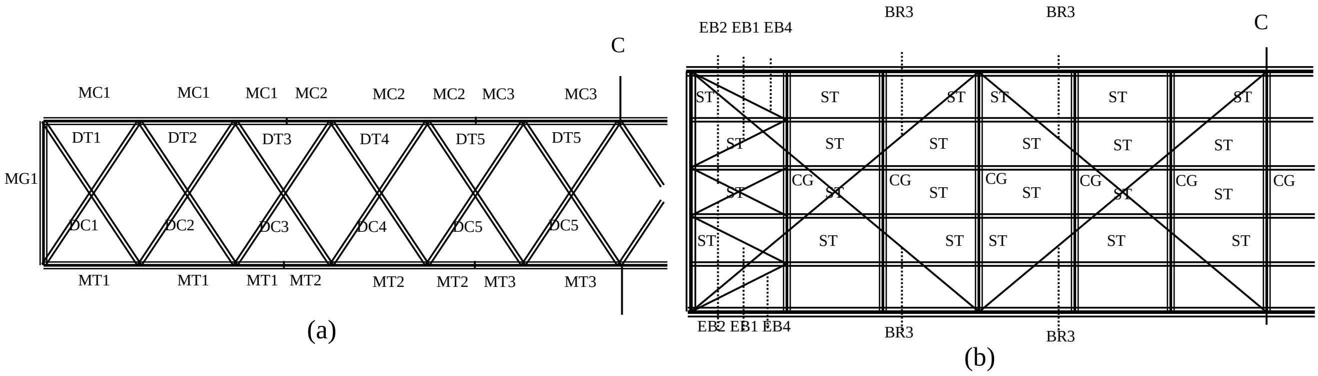

Fig. 2. Member set categorization: (a). Main truss girder, (b). Horizontal bridge deck [3]

entitled “member set” by considering similar cross sectional properties as shown in Fig. 2. Details of trains carried by the bridge and their frequencies illustrate that the bridge is subjected to variable amplitude loading [23].

The laboratory testing concluded that the bridge super structure material is wrought iron and the obtained values for elastic modulus, yield strength, ultimate strength in tension, fatigue strength and density are 195 GPa, 240 MPa, 383 MPa, 155 MPa and 7600 kg/m^3 respectively [3].

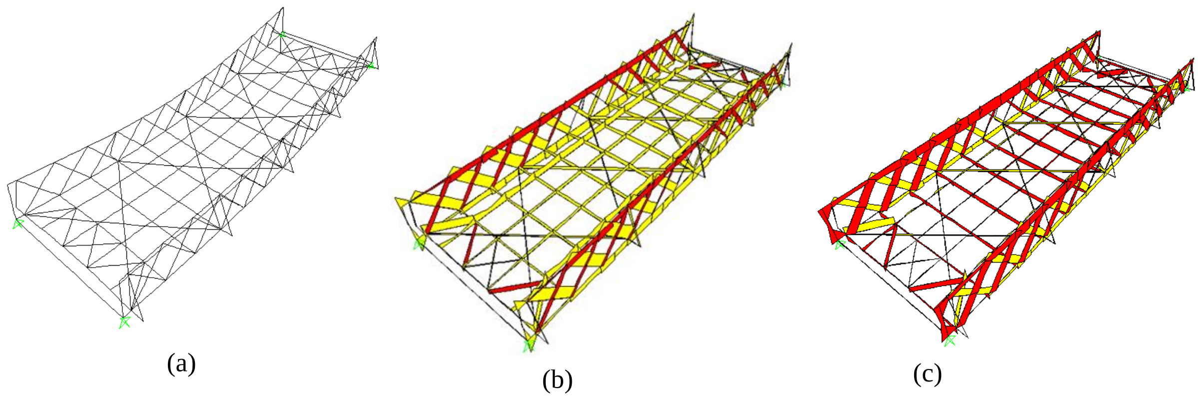

Fig. 3. The FE analysis results for moving train load: (a) Vertical displacement when the train is in the middle of the bridge (b) Maximum stress taken over all stress points at each cross sections when train is in the middle of the bridge (c) Minimum stress taken over all stress points at each cross sections when train is in the middle of the bridge cross sections when the train just before leave the bridge
Yellow color: Tensile stress- Red color: compressive stress

4 STRUCTURAL ANALYSIS

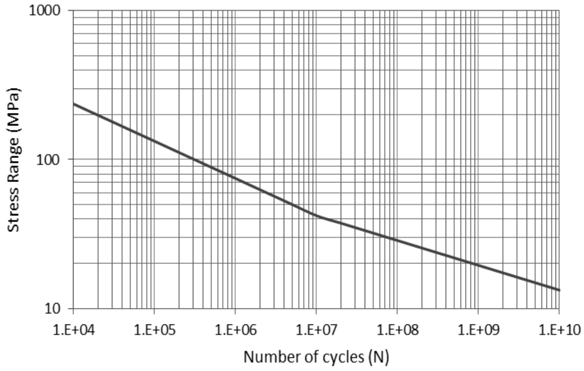

Fig. 4. S-N curves for WI-rivet detail

The bridge deck was analyzed using the general-purpose software SAP 2000. A three-dimensional (3D) model (Fig. 3) of one complete middle span of the bridge was analysed under actual loading to determine stresses in members and deflections, as well as variations of stresses under moving loads. The material properties recorded in section 3 and calculated section properties were utilized for this analysis. The bridge deck was modelled with 3D frame elements and the riveted connections were assumed to be fully-fixed [5]. Dynamic analysis was conducted for each different past and present passage of trains specified by the owner. The validation of FE model was done by comparing the results of time history dynamic analysis with those from measured time histories during the structural appraisal in year 2001 [3, 23]. These comparisons show that there are good agreement among analytical results of the FE model and the measurement of the actual bridge. Finally, the model is used to obtain past and present stress histories of each members due to each passage of trains.

5 FATIGUE STRENGTH

The *S-N* curve of detail category (also referred to as detail class) are generally used with nominal stress histories to capture the fatigue damage due to the local stresses near the connection. The detail category is determined by considering the current condition of the connection.

Field investigations revealed that the riveted wrought iron connections of the bridge represent lapped or spliced connection behaviour with normal clamping force. Therefore, riveted connections were classified as WI-rivet (i.e. WI-rivet detail category or class), which is proposed by the UK railway assessment code [1,24]. The different mean and design *S-N* curves for WI-rivet detail class have been proposed by previous researchers based on results of experiments on full scale riveted members [1]. The above design *S-N* curve of WI-rivet detail (i.e. mean minus two standard deviations, which has 2.3% probability of failure), which is shown in Fig. 4, was used for fatigue reliability assessment of this bridge. The corresponding slopes of *S-N* fatigue curve m_1, m_2, fatigue detail coefficients A_1, A_2 and constant amplitude fatigue threshold *CAFT* are 4,6, 3.117×10^{13}, 5.489×10^{16} and 42MPa respectively.

6 FATIGUE RELIABILITY ASSESSMENT

The stress ranges and average number of cycles per day at each members were calculated for each period using the rainflow counting algorithm. The stress range histograms for critical members and its probability density functions are plotted as shown Fig. 5. The Fig. 5 illustrates that the stress ranges of almost all critical members follow the log-normal distribution. Hence equivalent constant amplitude stress ranges (S_{re}) for each critical members of each member sets were calculated by Eq (5).

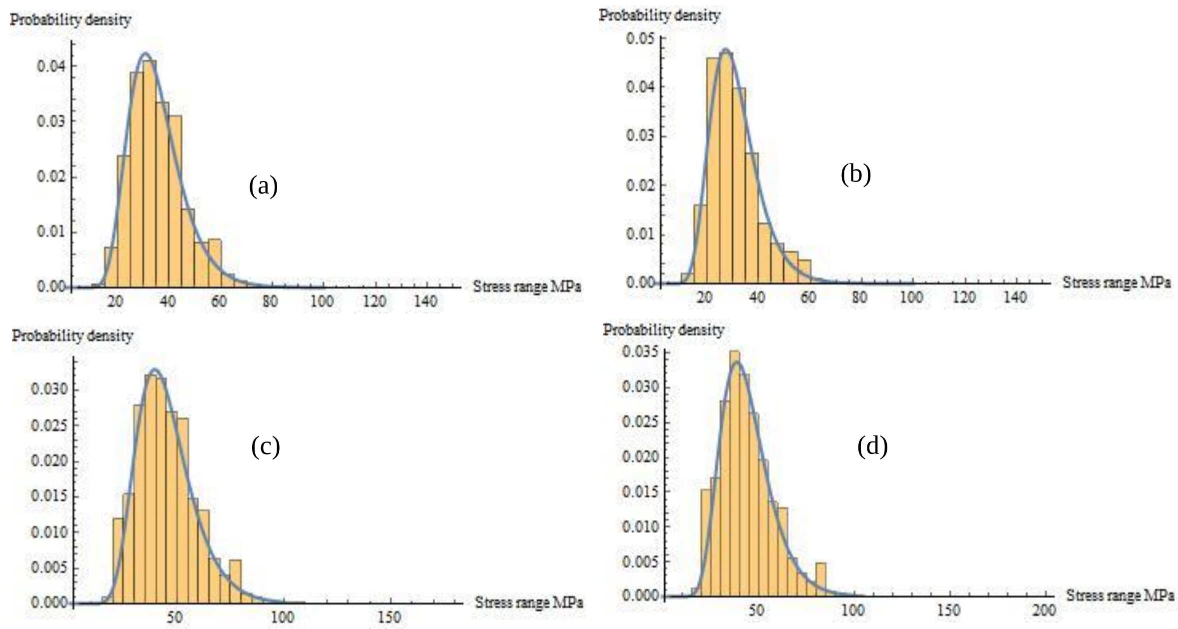

Fig. 5. Stress range histogram and its probability distribution function: (a) for critical member in cross girder set CG; (b) for critical member in stringer set ST; (c) for critical members in main girder set MT2; (d) for critical members in truss diagonal set DT3

The COV of S_{re} is considered as 0.1 [15,25]. The parameter A_1 and Δ are random variables and corresponding COV's are 0.45 and 0.3 respectively as discussed in section 2 [18,25]. Other parameters such as m_1, m_2, $CAFT$ and $N(t)$ are considered as the deterministic parameters. As all the random variables follow the log normal distribution, based on Eqs. (2) and (3), fatigue reliability index, β can be derived as follows:

$$. \beta(t) = \begin{cases} \dfrac{\lambda_\Delta + \lambda_{A_1} - m_1.\lambda_{S_{re}^L} - \ln N(t)}{\sqrt{\zeta_\Delta^2 + \zeta_{A_1}^2 + (m_1.\zeta_{S_{re}^L})^2}} & for\ N(t) \le \dfrac{A_1}{CAFT^{m_1}} \\ \dfrac{\lambda_\Delta + \lambda_{A_2} - m_2.\lambda_{S_{re}^B} - \ln N(t)}{\sqrt{\zeta_\Delta^2 + \zeta_{A_2}^2 + (m_2.\zeta_{S_{re}^B})^2}} & for\ N(t) > \dfrac{A_1}{CAFT^{m_1}} \end{cases} \quad (6)$$

where λ and ζ are lognormal parameters of the various random variables.

The cumulative number of cycles $N(t)$, lognormal parameters of S_{re} , A_1, A_2 and Δ are substituted to Eq. (6) and hence the fatigue reliability profiles (i.e. variation of fatigue reliability index with the age of the bridge) of the critical members of each member set of the bridge are generated and plotted in Fig.6. A target reliability index is defined to evaluate probability of limit state failure and corresponding fatigue life. Based on survival probability of 95%, target reliability indices was calculated as 1.65. The calculated fatigue lives are shown in Table 1.

The sequential law associated method [22], obtained nominal stress ranges in section 4 were used together to obtain remaining fatigue lives of critical members of each critical member sets of the bridge. The obtained fatigue lives of fatigue critical members of each member sets (i.e. which are possible to fatigue damage) are shown in Table 1. It is assumed that future sequence of passage is similar to that of the present day.

7 CONCLUSIONS

A probabilistic fatigue assessment approach and a deterministic approach consisting of a new damage indicator, which capture the loading sequence effect of variable amplitude loads more precisely than Miner's rule, were introduced to assess the fatigue life of an ageing railway

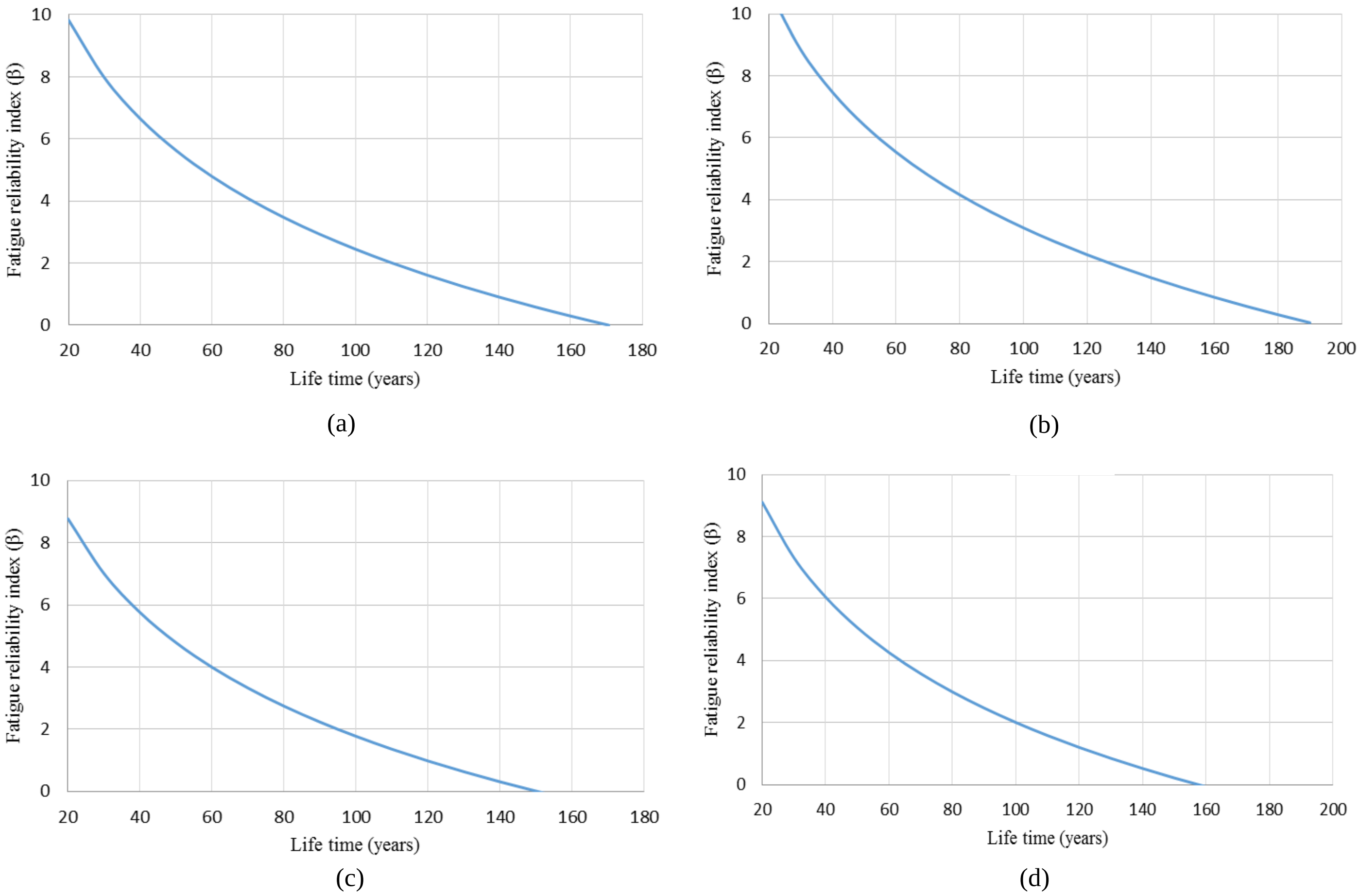

Fig. 6. Fatigue reliability index versus life of the bridge: (a) for critical member in cross girder set CG, (b) for critical member in stringer set ST, (c) for critical member in main girder set MT2, (d) for critical member in truss diagonal set DT3

bridge. Obtained fatigue lives were compared for critical members of each member sets as shown in Table 1. The Table shows that both deterministic and probabilistic approaches provide almost closer fatigue lives for bridge deck members (i.e. cross girders CG and stringers ST). However, it is opposite for the main girder truss members (i.e. main girder chords and truss diagonals).

Table shows that highly stressed member of main girder bottom chord MT2 is the most vulnerable to fatigue failure and the vulnerable members are located in the main girder consisting of truss members. Further, it seems that there are no more remaining lives for majority of vulnerable members of main girder truss under the 95% of survival probability. However, bridge is still in service without any recorded damage or fracture. The deterministic approach

Table 1. Summary of fatigue lives for critical members of each member sets

Bridge component	Member set	Fatigue life (years)	
		Deterministic Approach	Probabilistic Approach
		Damage stress model	β_{target}=1.65
Cross girders	CG	133	119
Stringers	ST	134	135
Main girder bottom chord	MT2	286	102
Truss diagonal (tension member)	DT3	259	108

predicts most vulnerable member for fatigue failure as the critical member in cross girder member set CG. According to the deterministic approach, the remaining fatigue life of the considered bridge is three more years.

The deviations of fatigue lives of both approaches illustrate that introduced probabilistic fatigue assessment approach may not preciously capture the loading sequence effect. However, it can be concluded that application of introduced probabilistic model provides a conservative fatigue assessment for railway bridges. Therefore, it is doubtful to conclude that this introduced probabilistic model and corresponding modal parameters provides a precise remaining life for ageing railway bridges.

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