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Fatigue reliability assessment of ageing railway truss bridges: Rationality of probabilistic stress-life approach

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ABSTRACT

Rail authorities all over the world are paying attention to extend the service lives of railway bridges. The famous 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. This comparison intends to conclude the possibility of capturing uncertainty behind loading sequence effect by proposed probabilistic fatigue assessment approach. Initially the paper presents the both approaches. Then the proposed approaches are applied to predict the fatigue lives of an ageing railway bridge. Finally predicted fatigue lives are compared and rationality, significance and validity of the proposed approaches are discussed.

1. Introduction

Majority of the railway bridges in the world are exceeding their design lives and bridge authorities are working on precise 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 is mainly done by either deterministic or probabilistic approach. Most of 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].

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The probabilistic fatigue assessments are 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] has proposed a probabilistic fatigue assessment methodology for riveted railway bridges under historical and present-day train loading. Kwon and Frangopol [18] have performed fatigue reliability assessment of steel bridges using the probability density function (PDF) of the equivalent stress ranges obtained by filed measurement data. Ni et al. [19] has 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] have 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 above 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 have 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 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. This comparison provides an indication of rationality of probabilistic stress-life fatigue approach for ageing railway bridges.

2. Fatigue reliability assessment using stress-life approach

This section introduces a precise probabilistic fatigue assessment approach and a recently proposed deterministic fatigue assessment approach. The first approach generally is based on probability of fatigue failure associated reliability index. The second approach is based on a new damage indicator, which captures the loading sequence effect more preciously than the Miner's rule.

2.1. Fatigue reliability index

This section proposes a method to determine fatigue reliability index of bridges based on probabilistic bilinear *S-N* approach. The fatigue reliability of a structural component or a detail category is related to the probability of not violating a particular fatigue limit state. Based on the limit state function (i.e. $g(t) = R - S$), the failure probability of a structural member or a detail category is defined as $P_f = P(g(t) < 0)$.

The reliability index provides a measure of fatigue damage of the considered detail category of the bridge. In other word, reliability index defines the probability of violating fatigue limit state. The fatigue reliability index is defined as,

$$\beta = \phi^{-1}\left(1 - P_f\right) \quad (1)$$

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

$$g(t) = \Delta - D \quad (2)$$

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,

$$D = \begin{cases} \dfrac{N(t)}{A_1}(S_{re}^{L})^{m_1} & for N(t) \le \dfrac{A_1}{CAFT^{m_1}} \\ \dfrac{N(t)}{(CAFT^{m_2-m_1} \times A_1)}(S_{re}^{B})^{m_2} & for N(t) > \dfrac{A_1}{CAFT^{m_1}} \end{cases} \quad (3)$$

where S_{re}^{L} and S_{re}^{B} 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 the 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 $A_2 = A_1 CAFT^{m_2-m_1}$, 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],

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

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 $\sum \left(n_i^o\right) + \sum \left(n_j^o\right)$ 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],

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

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

2.2. Damage stress model: a new damage indicator

This section introduces a new damage model consisting of a new indicator which predicts fatigue life in deterministic way. The definition of the damage indicator, D_i and the detailed description of the damage stress model for variable amplitude loading are given in the corresponding papers [3,22]. Also the accuracy of both damage stress model (DSM) and procedure of fully known *S-N* curve determination in fatigue life prediction have been confirmed by comparing the theoretical fatigue lives (i.e. obtained by Miner's rule and new DSM) with experimentally observed fatigue lives for few material types [3,22]. The concept is only summarized in this paper with an algorithm for comprehension.

For instance, a member is subjected to certain stress amplitude or stress range of σ_i for n_i number of cycles at load level i and N_i is the fatigue life (failure number of cycles) corresponding to σ_i (Fig. 5). Hence, the residual life at load level i can be obtained as $(N_i - n_i)$. The stress $\sigma_{(i)eq}$ which corresponds to the failure life $(N_i - n_i)$ is named as i^{th} level damage stress amplitude or stress range (otherwise it can be introduced as stress amplitude or stress range relevant to the residual life). Hence, the new damage indicator, D_i is stated as,

$$D_i = \frac{\sigma_{(i)eq} - \sigma_i}{\sigma_u - \sigma_i} \tag{6}$$

where σ_u is the intercept of the *S-N* curve with the ordinate at one-quarter of first fatigue cycle. Furthermore, it can be stated that, σ_u is the ultimate tensile strength amplitude or the range for rotating bending test-based *S-N* curves and it is the ultimate shear strength amplitude or the range for torsional fatigue test-based *S-N* curves.

Same damage is then transformed to load level $i+1$ and hence damage equivalent stress at level $i+1$ is calculated with the relation,

$$D_i = \frac{\sigma_{(i)eq} - \sigma_i}{\sigma_u - \sigma_i} = \frac{\sigma'_{(i+1)eq} - \sigma_{i+1}}{\sigma_u - \sigma_{i+1}} \tag{7}$$

Further simplification of Eq. (7),

$$\sigma'_{(i+1)eq} = D_i(\sigma_u - \sigma_{i+1}) + \sigma_{i+1} \tag{8}$$

where$\sigma'_{(i+1)eq}$ is the damage equivalent stress amplitude or the stress range at the level $i+1$. Thus the corresponding equivalent number of cycles to failure $N'_{(i+1)R}$ can be obtained from the *S-N* curve as shown in Fig. 1. The σ_{i+1} is the amplitude or the range of applied stress at the level $i+1$and when it is subjected to $n_{(i+1)}$ number of cycles, the corresponding residual life at the load level $i+1$, $N_{(i+1)R}$ is calculated as,

$$N_{(i+1)R} = N'_{(i+1)R} - n_{(i+1)} \tag{9}$$

Hence the damage stress amplitude or the stress range $\sigma_{(i+1)eq}$,which corresponds to $N_{(i+1)R}$ at load level $i+1$, can be obtained from the *S-N* curve as shown in Fig. 1. Then the cumulative damage at load level $i+1$ is defined as,

$$D_{(i+1)} = \frac{\sigma_{(i+1)eq} - \sigma_{i+1}}{\sigma_u - \sigma_{i+1}} \tag{10}$$

At the first cycle, the damage stress amplitude or the range $\sigma_{(i)eq}$ is equal to applied stress σ_1 and corresponding damage indicator becomes $D_i = 0$. Similarly at the last cycle, the damage indicator becomes $D_i = 1$ when $\sigma_{(i)eq}$ is equal to σ_u. Therefore, the damage indicator is normalized to one ($D_i = 1$) at the fatigue failure of the material and the same procedure is followed

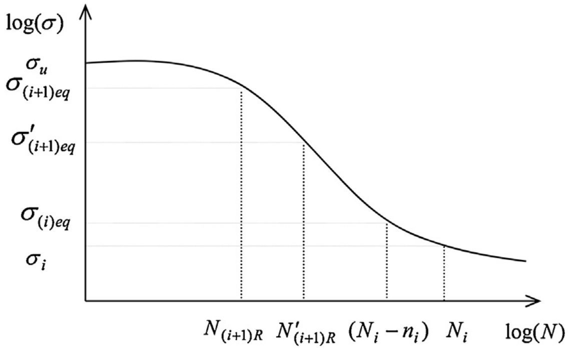

Fig. 1. Schematic representation of parameters in full range *S-N* curve.

Fig. 2. General views of considered bridge.

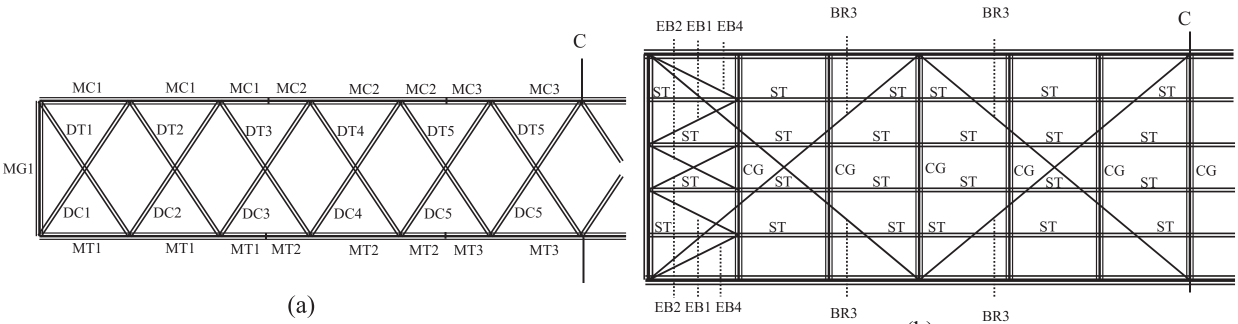

Fig. 3. Member set categorization: (a) main truss girder, (b) horizontal bridge deck [3].

until $D_i = 1$. Here, the defined fatigue failure is the time taken for the occurrence of the first through-thickness crack at the location of maximum stress of the structural component.

The above new damage indicator which consisted of DSM has been validated against fatigue testing data of few materials [3,22,23] and it has been confirmed that DSM provides closer prediction to fatigue testing data than Miner's prediction.

3. Case study- fatigue reliability assessment of an ageing railway bridge

The fatigue reliability of an ageing railway bridge is discussed in this section. The assessment were performed using introduced new models in Section 2.

3.1. Considered railway bridge

The selected bridge is one of the longest railway bridges in Sri Lanka spanning 160 m (Fig. 2). 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 entitled "member set" by considering similar cross sectional properties as shown in Fig. 3. Details of trains carried by the bridge and their frequencies illustrate that the bridge is subjected to variable amplitude loading [24].

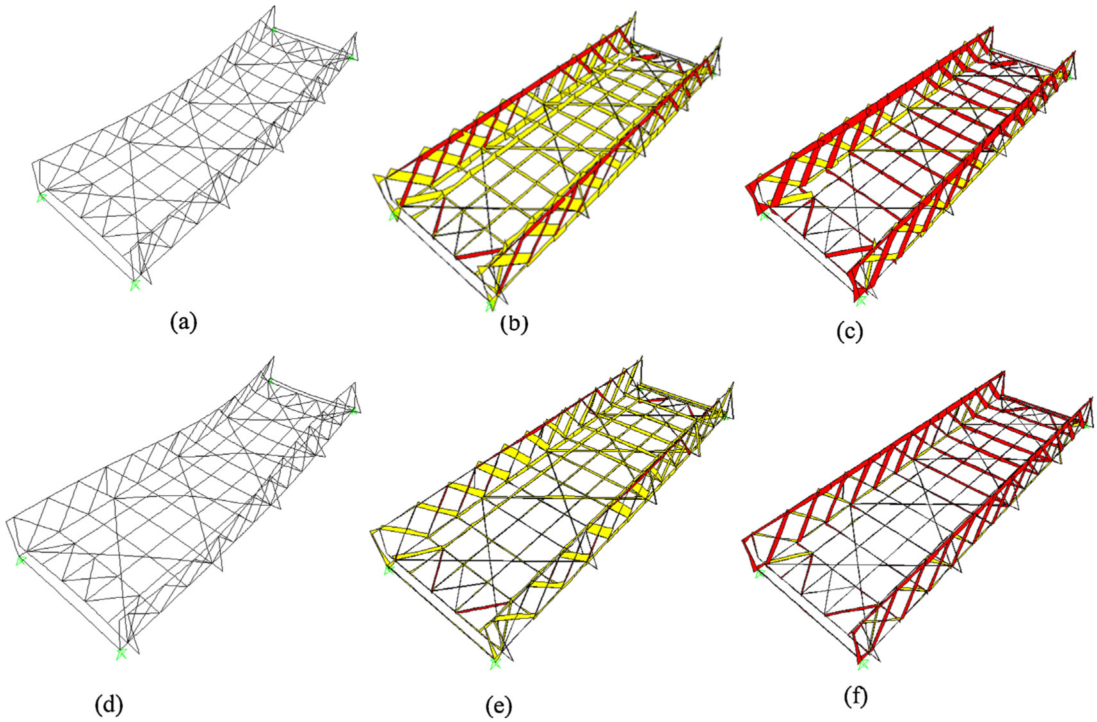

Fig. 4. The FE analysis results for moving train load: (a) vertical displacement when the train is in the middle of the bridge (b) maximum stress taken over all stress points at each cross sections when train is in the middle of the bridge (c) minimum stress taken over all stress points at each cross sections when train is in the middle of the bridge (d) vertical displacement when the train just before leave the bridge (e) maximum stress taken over all stress points at each cross sections when the train just before leave the bridge (f) minimum stress taken over all stress points at each cross sections when the train just before leave the bridge. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)
Yellow color: tensile stress
Red color: compressive stress

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].

3.2. Structural analysis

The bridge deck was analysed using the general- purpose software SAP 2000. A three-dimensional (3D) model (Fig. 4) 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 3.1 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 [3,24]. 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 nominal stress histories of each members due to each passage of trains.

Shrinkage of the free rivets are mostly restricted by the connected plates, which consequently are compressed through the thickness. The residual tensile force in the rivet and the compressive force in the plates get balanced each other; i.e. called as clamping force. Therefore the clamping force from the rivet generates a complex tri-axial stress state in the connected plate in the vicinity of the rivet hole [25]. Finally, the clamping force seems to be affected by the mechanism of distribution of stresses along the connection. The experience from the field practice reveals that resulting clamping force could vary substantially due to normal conditions. Therefore, it could consequently not be given a reliable value. Furthermore, it can be assumed a certain relaxation of the rivet clamping force due to creep, fretting of the interfacing plate surfaces, overloading (due to residual plastic deformation) and etc with the time (when bridge is ageing). Therefore, rivets are conservatively assumed to behave as non-preloaded bolts in ordinary clearance holes. Hence, net cross section, where the rivets are located, is considered for determining nominal stress histories of each members due to each passage of trains [26].

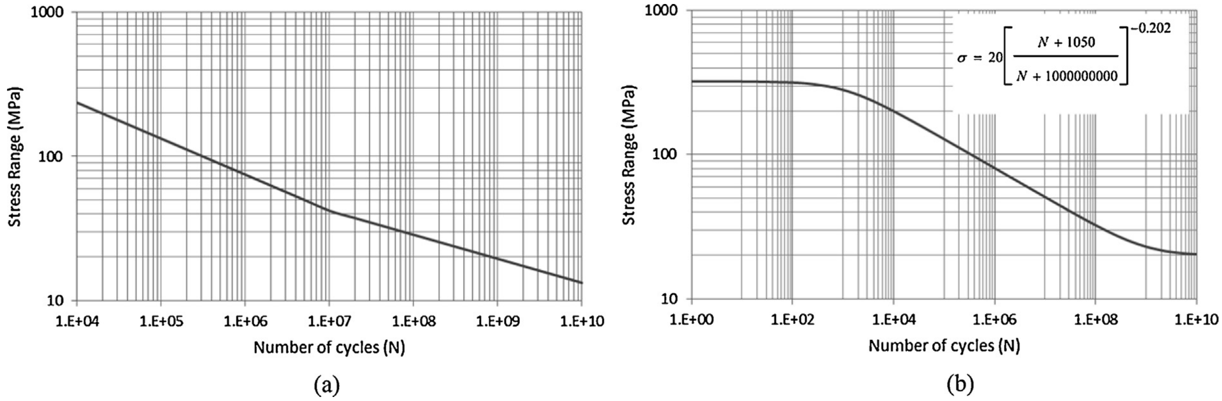

Fig. 5. *S-N* curves for WI-rivet detail: (a) curve given in UK railway assessment code (b) predicted curve for full range of number of cycles.

3.3. Stress-life fatigue strength curves

Secondary stress (local stress concentration) effect in riveted connection between the primary members of bridges was found to be one of the main reasons for fatigue damage of ageing steel bridges. Further it has been identified that the rotational fixity of riveted connection and the variation in the clamping force of rivets [25] are the major causes leading to fatigue cracking in riveted connection [1]. Therefore, the *S-N* curve of detail category (also referred to as detail class) are generally used with the nominal stress histories to capture the fatigue damage due to the above-mentioned stress concentration near the riveted connection. This detail category based *S-N* curves are obtained by modifying material *S-N* curve by corresponding SCF or by results of experiments on full scale riveted members. The detail category is determined by considering the quality of the workmanship and the current condition of the riveted connection.

Field investigations revealed that the riveted wrought iron connections of the bridge represent lapped or spliced connection behaviour with the 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 [1,27]. The different mean and design *S-N* curves for WI-rivet detail class have been proposed by previous researchers based on the results of experiments on full scale riveted members [1]. The above design *S-N* curve of the WI-rivet detail (i.e. mean minus two standard deviations, which has 2.3% probability of failure) was used for fatigue reliability assessment of this bridge. The corresponding slopes of *S-N* life fatigue curve m_1, m_2, the 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 42 MPa respectively.

Hence both *S-N* curves above were transferred to full range *S-N* curves using the previously proposed method in literature [3]. The obtained functions and the geometrical shapes of the curve given in UK railway assessment code and the predicted curve for full range of number of cycles are illustrated separately in Fig. 5.

4. Fatigue reliability assessment

The stress ranges and the average number of cycles per day at each members were calculated for each period using the rainflow counting algorithm. The member, which is most vulnerable to fatigue damage, is named as critical member in considered member set (i.e. shown in Fig. 3). The stress range histograms for the critical members and its probability density functions are plotted as shown in Fig. 6. The Fig. 6 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).

The COV of S_{re} is considered as 0.1 [15,28]. The parameter A_1 and Δ are random variables and corresponding COV's are 0.45 and 0.3 respectively as discussed in Section 2.1 [18,28]. 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:

$$\beta(t) = \begin{cases} \dfrac{\lambda_\Delta + \lambda_{A_1} - m_1 \times \lambda_{S^L_{re}} - \ln N(t)}{\sqrt{\zeta^2_\Delta + \zeta^2_{A_1} + (m_1 \times \zeta_{S^L_{re}})^2}} & for\ N(t) \le \dfrac{A_1}{CAFT^{m_1}} \\ \dfrac{\lambda_\Delta + \lambda_{A_2} - m_2 \times \lambda_{S^B_{re}} - \ln N(t)}{\sqrt{\zeta^2_\Delta + \zeta^2_{A_2} + (m_2 \times \zeta_{S^B_{re}})^2}} & for\ N(t) > \dfrac{A_1}{CAFT^{m_1}} \end{cases} \quad (11)$$

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

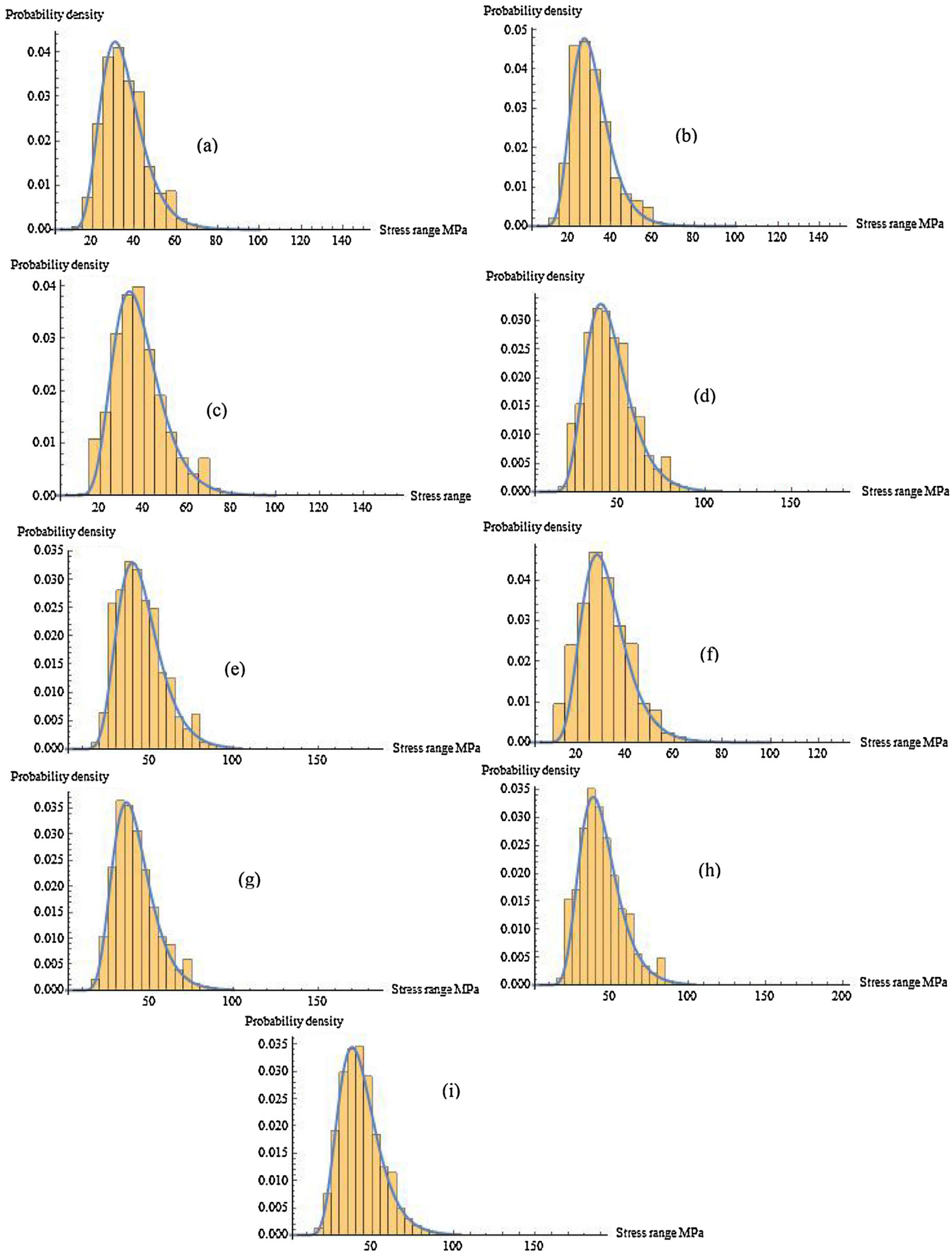

Fig. 6. Stress range histogram and its probability distribution function: (a) for critical member in cross girder set CG; (b) for critical member in stringer set ST; (c) for critical members in main girder set MT1; (d) for critical members in main girder set MT2; (e) for critical members in main girder set MT3; (f) for critical members in truss diagonal set DT1; (g) for critical members in truss diagonal set DT2; (h) for critical members in truss diagonal set DT3; (i) for critical members in truss diagonal set DT4.

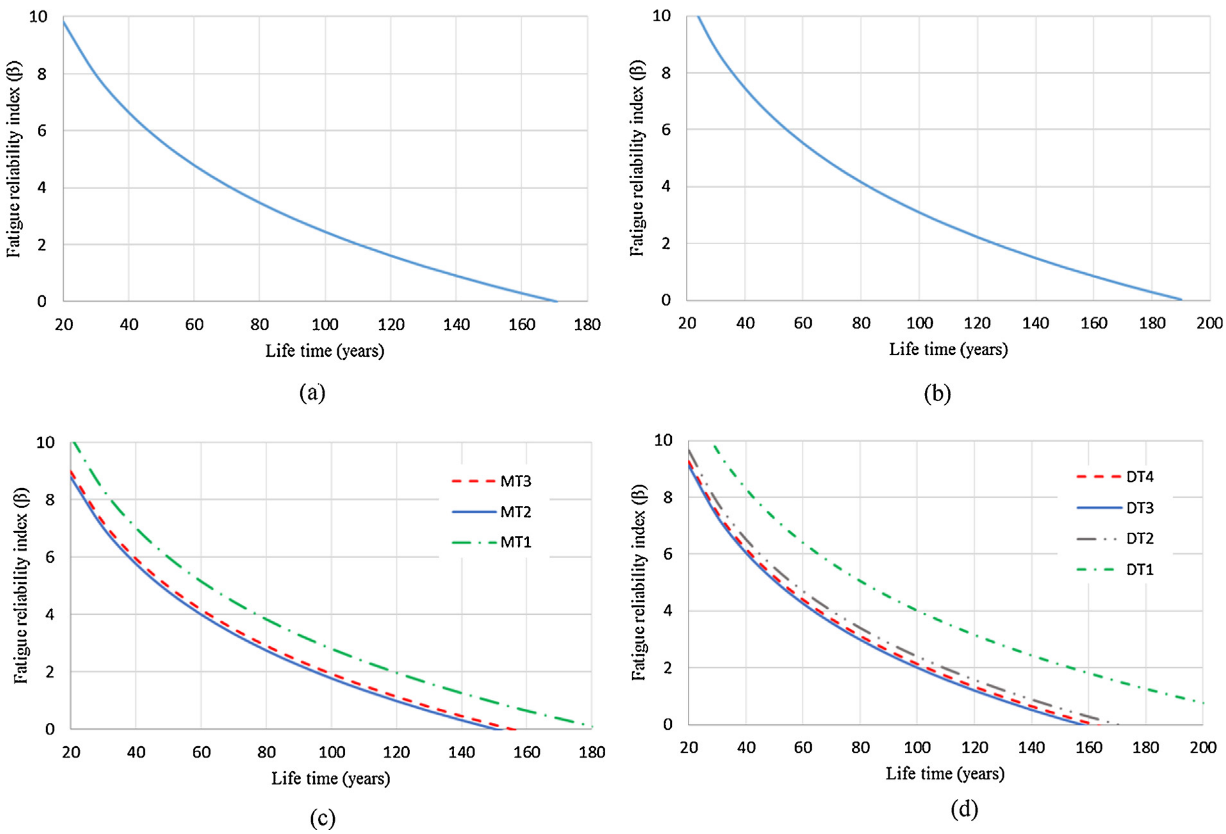

Fig. 7. Fatigue reliability index versus life of the bridge: (a) for critical member in cross girder set CG, (b) for critical member in stringer set ST, (c) for critical members in main girder set MT1, MT2 and MT3, (d) for critical members in truss diagonal set DT1,DT2,DT3 and DT4.

The cumulative number of cycles $N(t)$, lognormal parameters of S_{re}, A_1, A_2 and Δ are substituted to Eq. (11) 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. 7. A target reliability index is defined to evaluate probability of limit state failure (i.e. Eq. (2)) and corresponding fatigue life.

Referring to Eq. (1), for 5% of failure probability of limit state $g(t)$ in Eq. (2), table of the standard normal cumulative distribution function $\phi(\beta)$ gives a reliability index β as 1.65. That means a target reliability index of 1.65 is considered based on survival probability of 95% for fatigue failure probability of approximately 5% [18]. In similar way, for 50% of failure probability of limit state $g(t)$ in Eq. (2), table of the standard normal cumulative distribution function $\phi(\beta)$ gives a reliability index β as 0. That means a target reliability index of 0 is considered based on survival probability of 50% for fatigue failure probability of 50%. The zero value of target reliability index gives an indication of highest possibility of fatigue failure. Generally, target reliability level should be determined according to the importance levels of respective structural details [16,18]. Therefore, in this case study, two limits of target reliability index have been considered for more generalized fatigue life assessment. Those target reliability indices are 1.65 and 0, which corresponding to survival probability of 95% and 50% respectively as described above [18]. The calculated fatigue lives are shown in Table 1.

The sequential law associated proposed method shown in Section 2.2, obtained nominal stress ranges in Section 3.2 and full range *S-N* curves shown in Fig. 5 were used together to obtain remaining fatigue lives of critical members of each 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.

5. Discussion and conclusions

A probabilistic fatigue assessment approach and a deterministic approach consisting of a new damage indicator, which captures 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 bridge. Obtained fatigue lives were compared for critical members of each member sets as shown in Table 1. The Table shows that both the 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 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	$\beta_{target}=1.65$	$\beta_{target}=0$
Cross girders	CG	133	119	170
Stringers	ST	134	135	191
Main girder bottom chord	MT1	444	128	183
Main girder bottom chord	MT2	286	102	150
Main girder bottom chord	MT3	290	106	154
Truss diagonal (tension member)	DT1	312	165	235
Truss diagonal (tension member)	DT2	292	118	170
Truss diagonal (tension member)	DT3	259	108	157
Truss diagonal (tension member)	DT4	283	111	161

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 (i.e. MT2, MT3, MT1, DT2, DT3 & DT4) under the 95% of survival probability. Under the 50% survival probability, considered bridge has about 20 more years of remaining fatigue life. However, bridge is still in service without any recorded damage or fracture. The deterministic approach predicts the most vulnerable member for the 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 precisely 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 provide a precise remaining life for ageing railway bridges. Authors are currently paying their attention on experimental validation of the probabilistic approach and damage stress model predicted fatigue lives.

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