REMAINING SERVICE LIFE BASED MAINTENANCE STRATEGY FOR RAILWAY BRIDGES

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ABSTRACT**:** This paper proposes an essential maintenance strategy for riveted railway bridges based on an accurate remaining fatigue life estimation technique. The strategy mainly consists of structural appraisal based critical members and connections identification, remaining fatigue lives estimation of critical connections and time dependent member replacement or strengthening scheme. In the stage of critical member identification, detailed structural appraisal has to be conducted giving priority to visual inspection for condition, FE analysis, and material testing, experimental static and dynamic load testing. Accurate fatigue life estimation techniques of both members and connections consist of measured stress histories and recently developed new fatigue models. A verification of the proposed strategy is conducted by comparing the predicted replacement scheme with sequence of experimental failure and fatigue life of selected test specimens. Finally, the proposed strategy was applied to a case study railway bridge to obtain time dependent member replacement scheme and obtained results are compared with previous estimations. Hence, validity and merits of proposed strategy were confirmed.

KEYWORDS: maintenance strategy, fatigue life, riveted bridge, structural appraisal

1. INTRODUCTION

Today, many of the structures in the world are getting old and a very large existing stock of civil infrastructures is in need of maintenance, rehabilitation or replacement. Considering the management of structures in terms of maintenance, member replacement had a wide acceptance during this period (Kong et al. 2005). However, there are few studies regarding the member replacement based maintenance guidelines for railway bridges. Most of them are especially based on rating factors against failure (rating of load carrying capacity). Since railway bridges are most liable to fail due to the effect of fatigue, remaining fatigue life based maintenance strategies illustrate more assurance than rating factor based policies (Netwok rail, 2001). However, very few studies have only been concentrated to remaining fatigue life based member replacement strategy. There the remaining fatigue life estimation technique is generally based on combination of specification load given stress histories, Miner's rule and design fatigue curve. But the experiences from engineering practices have indicated that fatigue analysis based on specification loads and distribution factors usually underestimates the remaining fatigue life of existing bridges by overestimating the live load stress ranges (Spyrakos et al. 2004). Further the Miner's rule does not properly take account of loading sequence effect (Suresh, 1998; Siriwardane et al. 2008). Therefore it is doubtful to utilize the remaining fatigue life based available strategies for railway bridges.

To overcome this problem, this paper proposes an essential maintenance strategy for riveted railway bridges based on an accurate remaining fatigue life estimation technique. Initially paper describes the proposed strategy in detail. Then the verification of the proposed strategy is confirmed. Finally, proposed strategy is applied to a case study railway bridge to obtain time dependent member replacement scheme. The obtained results are compared with previous estimations.

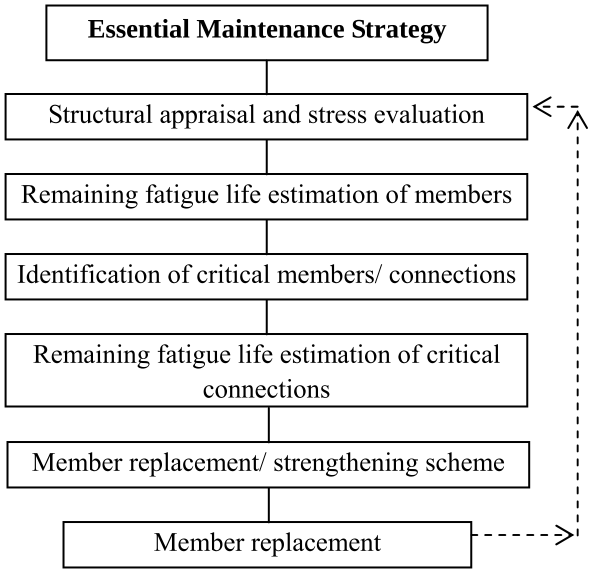


Fig. 1. Flow chart of proposed maintenance strategy

2. PROPOSED MAINTENANCE STRATEGY

Proposed maintenance strategy for riveted railway bridges is discussed in this section. Especially this is a member replacement based essential maintenance strategy. It consists of two major parts such as identification of critical members/connections and member replacement/strengthening scheme. The identification procedure of the critical members is dependent on the remaining fatigue life of each member since railway bridges are most liable to fail due to the effect of fatigue. The proposed remaining fatigue life estimation technique is specially depend on structural appraisal based stress evaluation and recently developed damage model. Flow chart of the proposed strategy is shown in Fig. 1.

2.1 Structural appraisal and stress evaluation

In remaining fatigue life estimation, it is essential to determine the stress ranges generated by the passage of trains over the bridge. Therefore, it is required to know the stress cycles of all the critical members for trains which are included in present and past timetables.

Initially a condition survey has to be carried out to assess the present geometric condition and damages. Generally, it consists of detailed visual examination, in-situ measurements of each component of the bridge and non-destructive field examinations. Then laboratory tests are carried out to determine the current mechanical properties and chemical composition of the bridge materials. Then static and dynamic load testing is recommended as the next major step to study the real behavior of the bridge under various load combinations. The obtained results are used to develop a proper analytical model and further assists in evaluating actual dynamic factors of each structural component. Finally the bridge is subjected to finite element (FE) analysis under test and actual loadings to determine stresses and deflections, as well as variations of stresses under moving loads. Material properties which are obtained through laboratory tests and current geometric properties obtained from condition assessment are applied to the FE model for more realistic outputs. The validation of the FE model has to be done by comparing the results from analysis with those from field-tests. The FE model which gives better comparison to load test results is nominated as “validated analytical model”. Hence, validated analytical model is used to obtain past and

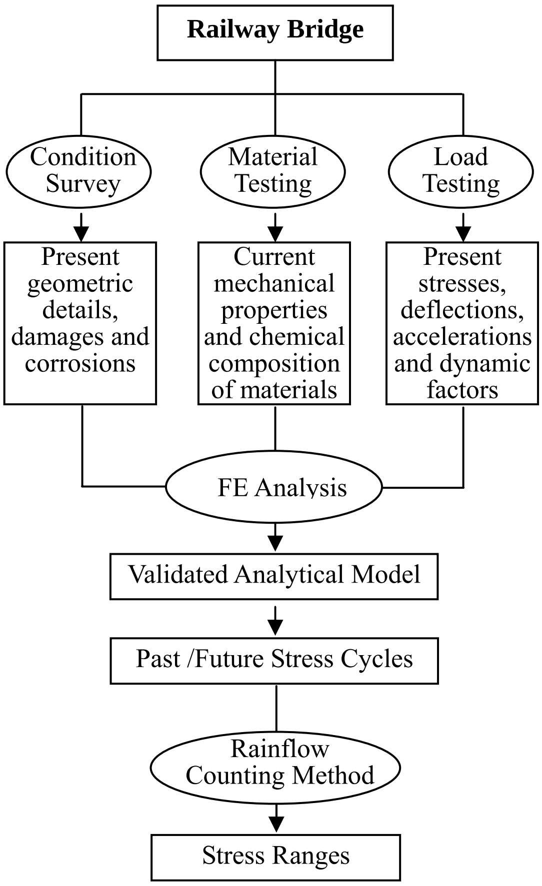


Fig. 2. Flow chart for the past /future stress evaluation method

present static stress histories due to passage of trains specified by the owner.

Due to the dynamic effect of moving trains, the actual working stresses should be higher than the analytical static stress. Therefore dynamic factors are used to multiply the static stress to get the service stresses of each member. Finally, the stress histories have to be converted into stress ranges using the rainflow counting method (Downing et al. 1982). The described stress evaluation method is briefly summarized as shown in Fig. 2.

2.2 Remaining fatigue life estimation of members

The recently developed sequential law (Mesmacque et al., 2005, Siriwardane et al., 2007 & 2008) is recommended to use for remaining fatigue life estimation, instead of Miner's rule, which was used as the fatigue theory in previous methods. A detailed description of the damage stress model and the definition of damage indicator, D_i is described in the corresponding paper (Siriwardane et al., 2007 & 2008). Here only the concept of new damage indicator is motioned for comprehension. Suppose a component is subjected to certain stress amplitude or stress range 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. 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 can be introduced as stress amplitude or stress range relevant to the residual life). Hence, the new damage indicator, D_i is stated as,

$$D_i = \frac{\sigma_{(i)eq} - \sigma_i}{\sigma_u - \sigma_i} \tag{2.1}$$

where σ_u is the intercept of the Wöhler 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 range for rotating bending test-based *S-N* curves and it is the ultimate shear strength amplitude or range for torsional fatigue test-based *S-N* curves.

At the first cycle the damage stress amplitude or range $\sigma_{(i)eq}$ is equal to applied stress σ_i and corresponding damage indicator becomes D_i=0. According to the proposed methodology, current damage has to be then transformed to next load level. Therefore, at 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 same procedure is followed 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. In the case of railway bridge components, it can probably be taken as the time taken for initiation of crack near a connection (rivet or bolt).

To capture the fatigue damage due to the secondary stresses near the riveted connection or discontinuities, detail class (BS 5400, 1980) of riveted connection based Wöhler curves are considered for life estimation. But, chosen fatigue curve only describes stress ranges, which are corresponding to more than ten thousands of failure cycles (usually called the partially known Wöhler curve). In the case of sequential law it is essential to use the Wöhler curve for full range of the number of cycles. Therefore, the chosen partially known Wöhler curve has to be transferred to fully known Wöhler curve by using Kohout and Vechet Wöhler curve modeling technique (Kohout et al. 2001).

2.3 Identification of critical members/ connections

The members which have the lowest remaining fatigue life of each member set (set of members which has the same load capacity) is called as the "critical members" in this study. Generally, these members are to be subjected more attention in the member replacement based maintenance. From the previously obtained remaining fatigue lives (section 2.2), it can be easily identify these critical members. The connections which are joining to the previously obtained critical members (section 2.2) are termed as "critical connections".

2.4 Remaining fatigue life estimation of critical connections

The stress concentration effect in connections between the primary members of bridges was found to be one of main reasons for fatigue damage (Imam et al. 2005; Fisher et al. 1980). Most of such connections are subjected to multiaxial fatigue. To capture this effect at riveted connections or discontinuities, detail class (BS 5400, 1980) of riveted connection based Wöhler curves are considered in previous life estimation. However, the variation of real rotational fixity, clamping force and geometry at the connection cause to change the real stress distribution at the connections. Such changes may results to give over or under prediction to estimated fatigue life (in section 2.2) of the corresponding member (which joined to the connection). As a result, replacement of members based on previously determined remaining lives (section 2.2) may not be an appropriate maintenance procedure. Therefore, replacement of members based on fatigue lives of critical connections is found to be a more appropriate strategy. This section describes the methodology to estimate remaining fatigue life of such connections.

Initially the all critical connections should investigate non-destructively to determine the current condition. This is conducts using different types of tests such as X-ray, ultrasonic, magnetic particle, radiographic examinations except to visual examination. The connections which do not illustrate significant change from the initial state or condition are not subjected any unexpected stress concentration. Other connections which have been subjected to significant change may have to asses in remaining fatigue life sense to determine the degree of criticality.

The remaining fatigue lives of the connections, which are not subjected to significant deviation from the initial condition, is finalized as same as the lowest remaining fatigue life of the member that jointed to particular connection. The remaining fatigue lives of other connections, where the conditions have been significantly changed, are to be evaluated based on the current geometric condition, secondary stress distribution and etc. Later part of this section describes multiaxial stress based fatigue life estimation procedure of riveted connections.

This procedure is specially based on newly proposed multiaxial fatigue model. The considered failure mechanism (damage process) is in mesoscopic scale and detailed descriptions are shown in authors' previous work (Siriwardane et al. 2008). Initially, the accumulated plastic strain per each stabilized cycle,

$$\varepsilon_s^{pc} = \frac{4}{\sqrt{3}} \frac{2k^* - k_{\max} - k_{\min}}{c} \tag{2.2}$$

has to be obtained from finite element employed secondary stress analysis of the connection or part of the member (sub model). Where $c = b + 2\eta$. The b and η are the mesoscopic linear hardening modulus and the shear modulus respectively. The k^* is the radius of the smallest hypersphere which contains the entire history of the macroscopic deviatoric stress amplitude of the stabilized cycle. The $k_{\max}$ and $k_{\min}$ are the maximum and the minimum values of mesoscopic yield stresses that can be reached during the loading cycle. Then the fatigue life is calculated from the new damage indicator,

$$D_i = \frac{(\varepsilon_s^{pc})_{(i)eq} - (\varepsilon_s^{pc})_i}{(\varepsilon_s^{pc})_u - (\varepsilon_s^{pc})_i} \tag{2.3}$$

where $(\varepsilon_s^{pc})_i$ is accumulated plastic meso-strain per stabilized cycle of n_i number of cycles at load level i. The N_i is the fatigue life (number of cycles to crack nucleation) corresponding to $(\varepsilon_s^{pc})_i$ can be estimated from,

$$N = A(\varepsilon_s^{pc})^{-\xi} \tag{2.4}$$

where ξ and A are material parameters to be determined from fatigue tests. The accumulated plastic meso-strain $(\varepsilon_s^{pc})_{(i)eq}$, which corresponds to the failure life (N_i–n_i) is named as i^{th} level damage accumulated plastic meso-strain. The $(\varepsilon_s^{pc})_u$ is the accumulated plastic meso-strain which corresponds to one-quarter of first fatigue cycle. According to the proposed methodology, current damage has to be then transformed to next load level. As similar o the previous sequential law in section 2.2, here also the damage indicator is normalized to one (D_i=1) at the fatigue failure of the material and same procedure is followed until D_i=1.

2.5 Member replacement/ strengthening scheme

The lowest remaining fatigue life of the connections describes the remaining fatigue life of the bridge. Once the age of the bridge reached to this value (when bridge life becomes zero), it is advisable to replace the corresponding critical member by new member with longer fatigue life. Same time, the associated connection is also to be recommended to strengthen. After this essential repair, it is recommended to obtain new sequence for future member replacement by following the similar procedure from the beginning. This type of maintenance strategies extends the service life of the (fatigue capacity) bridge in safest manner.

3. VARIFICATION OF PROPOSED STRATEGY

In this section, the experimental (real) failure life based member replacement sequence is compared with the proposed strategy predicted scheme of five specimen members. The experimental fatigue lives were obtained in the high cycle fatigue regime under variable amplitude combined bending and torsion loading (Marciniak et al. 2008). The comparison between the predicted member replacement scheme and the experimental results based scheme are shown in Table 1. In addition, it has been compared with two previous strategies based member replacement schemes as shown in Table 1. The first scheme is based on load rating factors (AASHTO, 1990; Bridge Diagnostics, 2002). The second strategy is based on remaining fatigue life estimation of members (Caglayan et al. 2009). Here, Miner's rule is considered as the fatigue law (damage model). The Table 1 shows that there is better agreement

Table 1. Comparisons of member replacement schemes

Designation of test specimen	Experimental		Previous strategy 1 (AASHTO, 1990)		Previous strategy 2 (Caglayan et al. 2009)		Proposed strategy	
	Service life (sec)	Sequence	Service life (sec)	Sequence	Service Life (sec)	Sequence	Service Life (sec)	Sequence
NWL 4	42795	1	27265	1	58243	1	49628	1
KWL 4	90705	2	44325	2	93497	2	78239	2
KWL 2	98948	3	68573	4	202229	5	116975	4
KWL 8	123275	4	158623	5	162519	4	109076	3
NWL 1	130630	5	48329	3	147265	3	123568	5

between the proposed strategy based sequence than the previous strategies based sequences. This verification reveals the validity of the proposed maintenance strategy in predicting the member replacement scheme in variable amplitude proportional loading conditions.

4. CASE STUDY: DETERMINATION OF MEMBER REPLACEMENT SCHEME FOR RAILWAY BRIDGE

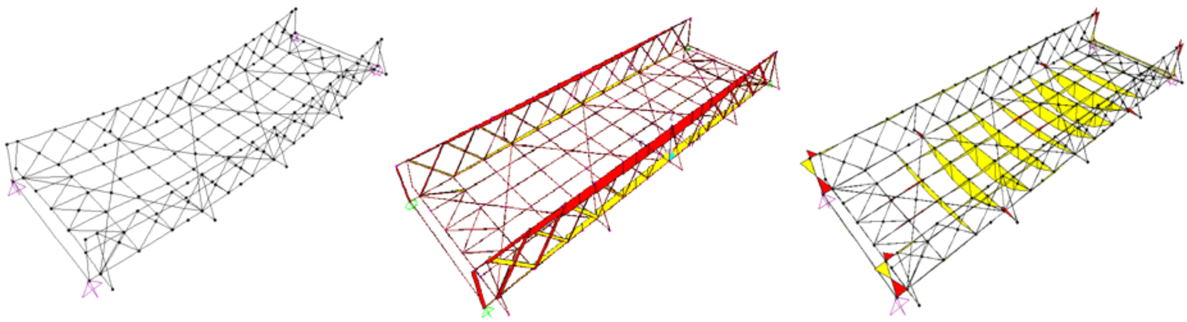
The selected bridge is one of the major railway bridges in Sri Lanka spanning 160 m (Fig.3). It is a six span-riveted bridge with double lane rail tracks having warren type semi through trusses, supported on cylindrical piers. The bridge deck is made of wrought iron and the piers are made of cast iron casings with infilled concrete. The bridge was constructed in 1885. The described strategy is used to determine the suitable member replacement scheme for the bridge.

4.1 Structural appraisal and stress evaluation

The condition survey reveals that no visual cracks were observed in any component of the super structure. In situ measurements of member sizes, connections and support bearings verified the fact that the existing drawings were applicable and only few significant variations were observed. The obtained values from material testing 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.

Static and dynamic load testing were performed to study the real behavior of the bridge under various load combinations. The in situ measurements were performed using two M8 engines, each weighting 1120 kN, which is the heaviest rail traffic in current operation. The bridge was instrumented with strain gauges, accelerometers and displacement transducers. The considered three static load combinations are defined as static load case (SLC) 1,2 and 3 by considering criteria of maximum shear effect, maximum bending effect (maximum deflection) and maximum torsion effect to the bridge deck respectively. The criteria, which were considered for dynamic load combinations, basically illustrate that

Fig. 3 General view and identified corroded locations of the bridge



(a) (b) (c)

Fig.4. 3D frame element model for single span (a) Deflected shape for SLC 2, (b) Axial force diagram at SLC 2, (c) Bending moment diagram at SLC 2

impact effect to the bridge with different levels of speed and traction force effect. Apart from the above mentioned formal field load testing, the bridge was subjected to a two days continuous field measurement program under present day actual traffic. The obtained sample measurements are shown in authors' previous publication (Siriwardane et al. 2007). Hence the dynamic factors were obtained as 1.3, 1.4 and 1.4 for main truss girders, secondary cross girders and stringers respectively.

The Bridge deck was analyzed using the finite element (FE) method employing the general-purpose package SAP 2000. A three-dimensional (3D) model (Fig. 4) of one complete middle span of the bridge was analyzed under test loading and actual loading to determine stresses in members and deflections, as well as variations of stresses under moving loads. The validation of FE model was done by comparing the results from analysis with those from field-tests as shown in Table 2. Designations of the members and connections of the bridge are schematically shown in Fig. 5. From the results of static load cases, it was seen that there is a good agreement among analytical results of the FE model and the measurement of the actual bridge. Hence, the considered 3D frame element model was defined as "validated analytical model".

Since the used types of trains changes with age of the bridge, the age had to be divided in to several periods. According to the time tables of the bridge, it could be decided that the traffic sequence is almost constant during a single day of each period of age. Then the validated analytical model was used to obtain the elastic stress histories of each critical member during a single day of each period. Due to the dynamic effect of moving trains, the actual

Table 2. Comparison of FE analytical results with load test results

Static load case	Displacement (mm)			Stress (MPa)		
	Location (Connection)	Load test	FEM	Location (Member)	Load test	FEM
SLC 1	TB7	19.4	21.0	TDC3	-40.2	-40.6
				TDT3	51.4	57.3
				TMB6	47.3	48.2
SLC 2	TB7	21.3	22.5	TDC3	-37.8	-37.7
				TDT3	44.5	43.6
				TMB6	53.5	53.9
SLC 3	TB7	-	19.1	TDC3	-39.5	-39.9
				TDT3	35.2	41.5
				TMB6	39.0	44.7

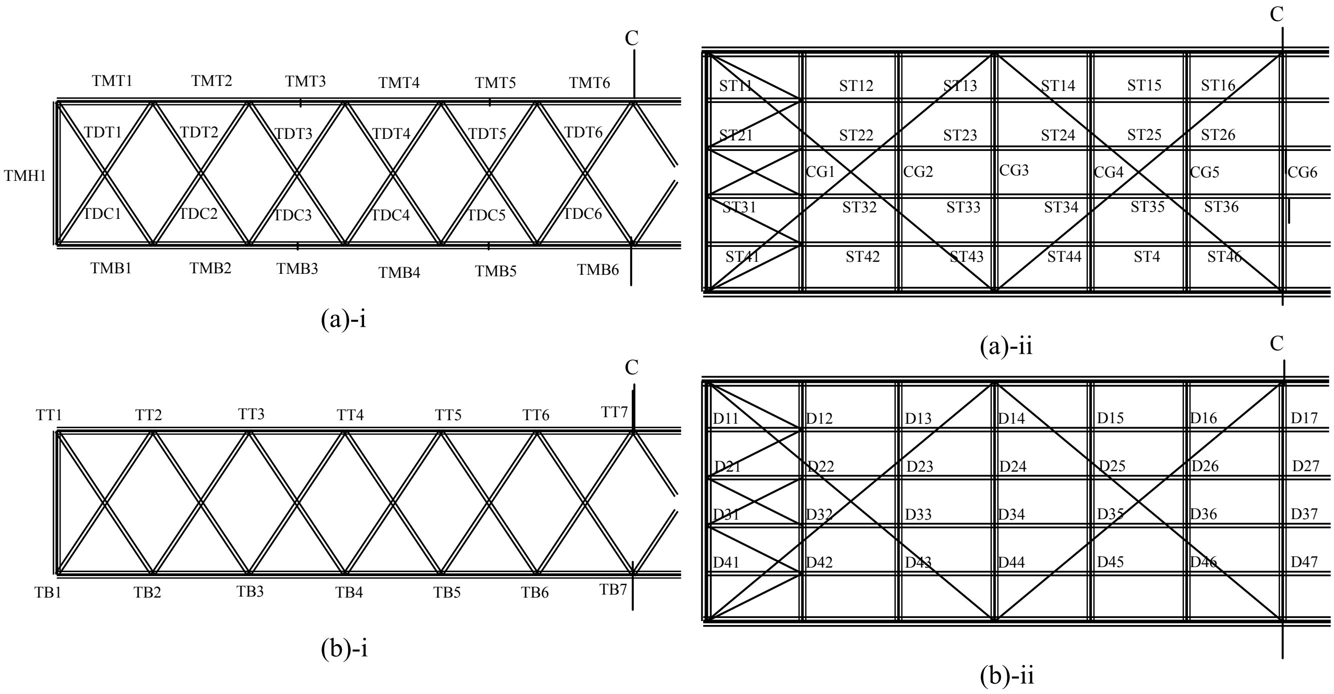


Fig. 5. Designations of (a) Members: i. Main truss girder, ii. Horizontal bridge deck; (b) Connections: i. Main truss girder, ii. Horizontal bridge deck

working stresses should be higher than the analytical static stress. Therefore, the experimentally found dynamic factor of each member was used to multiply the static stress to get the service stresses. Then the stress histories were converted into stress ranges using rainflow counting method.

4.2 Remaining fatigue life estimation of members

The proposed method (section 2.2) was used to calculate the remaining fatigue life of members. The riveted connections were classified as class Wrought-iron (WI), which is proposed by the UK railway assessment code (Network Rail 2001). Hence the *S*-*N* curve, which is mentioned under the UK railway assessment code for WI detail class connection (Network Rail 2001), was transferred to a fully known Wöhler curve using the method mentioned in section 2.2. The obtained function of the new fatigue curve is,

$$\sigma = 20\left(\frac{N+1050}{N+1000000000}\right)^{-0.202} \quad (4.1)$$

Assuming that future sequence of passage is similar to that of the present day, the remaining fatigue lives were calculated for all the members.

4.3 Identification of critical members/ connections

The identified critical members which exhibits the lowest remaining life of each member set are shown in Table 3. The corresponding critical connections are also tabulated in Table 3.

4.4 Remaining fatigue life estimation of critical connections

The critical connections (section 4.3) are subjected to both visual and X-Ray examinations. Hence, it was determined that TT 3 and TB 2, other all

Table 3. Remaining fatigue lives for critical members and corresponding critical connections

Designation of Members	Corresponding connections	Remaining Fatigue life (years)
TMB3	TB4	323
TMB5	TB6	165
TMB6	TB7	169
CG4,5	D15-45, D16-46	12
ST 16,26,36,46	D17-47	13
TDT1	TT1, TB2	191
TDT2	TT2,TB3	171
TDT3	TT3,TB4	138
TDT4	TT4,TB5	162

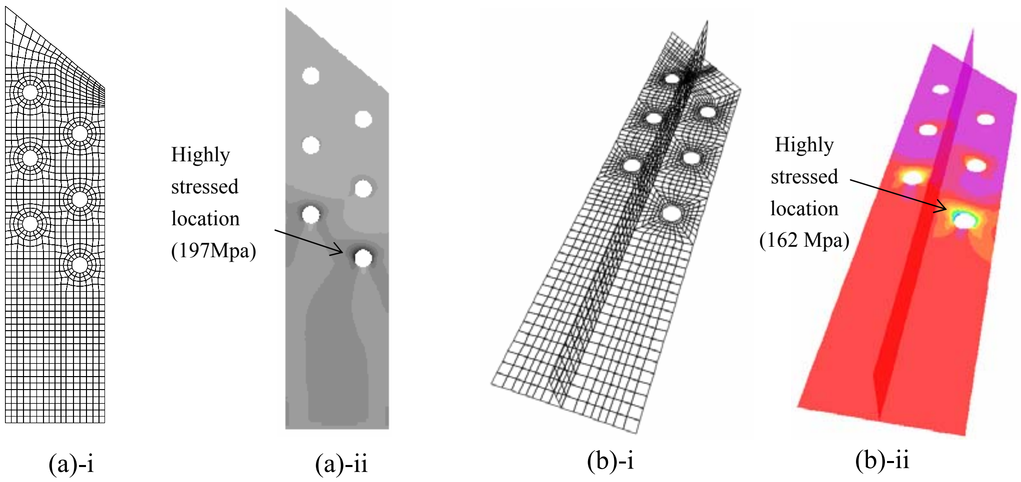


Fig. 6. The stress analysis at (a). TT 3 connection: i. Fine FE mesh, ii. Maximum von Mises stress contour when no clamping force at all six rivets, (b). TB 2 connections: i. Fine FE mesh, ii. Maximum von Mises stress contour when no clamping force at all six rivets

connection are not subjected to severe changes. However, it seems that clamping forces of some rivets at these two connections are significantly disappeared.

Therefore the fatigue damage is evaluated based on the state of stress due to release of clamping force while all the rivets of these two connections have no clamping force. Therefore, a critical member without rivets was considered for analysis (Fig. 6). The four-node shell elements were used for the finite element analysis. The actual air gap restraint conditions were considered in the model to represent unilateral contact between rivet and plate. The obtained maximum stress contours are shown in Fig. 6 for both TB 3 and 6 connections. This shows that stresses are operating bellow the yield limit of the material and highly stressed locations (critical

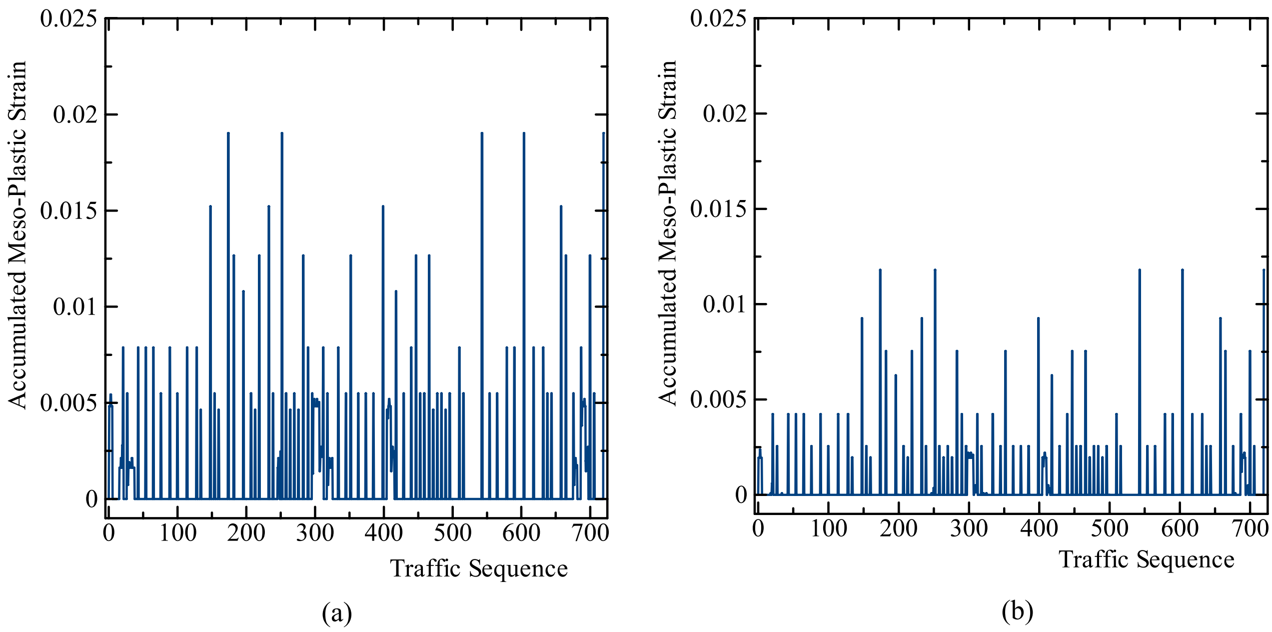


Fig. 7. Accumulated meso-plastic strain variation with the traffic sequence per single day at present, (a) for TT3 connection: (b). For TB2 connection

Table 4. Remaining fatigue lives for critical connections

Bridge component	Designation of critical connection	Corresponding critical member	Remaining fatigue life (years)
Main girder bottom chord	TMB3	TB4	323
Main girder bottom chord	TMB5	TB6	165
Main girder bottom chord	TMB6	TB7	169
Cross girders	CG4,5	D15-45, D16-46	12
Stringers	ST 16,26,36,46	D17-47	13
Truss diagonal (tension member)	TDT1	TT1, TB2	45
Truss diagonal (tension member)	TDT2	TT2,TB3	171
Truss diagonal (tension member)	TDT3	TT3,TB4	24
Truss diagonal (tension member)	TDT4	TT4,TB5	162

locations) are subjected to biaxial proportional state of stress. Assuming that single day traffic sequence (time table) is repeating every day, one day time history is considered as loading block in this study.

The obtained principle stress variations are transferred to the equivalent stabilized cycles, which give the same damage to the material, using rainflow cycle counting technique (Downing et al. 1982) and modified Goodman relation (Suresh, 1998). Material parameters $\alpha, \beta, \gamma, \xi$, c and A for Wrought iron were calculated as 1.0832, 15 MPa, 0.0473, 1.36, 12923 MPa and 613 respectively. Hence, the accumulated plastic meso-strain per each stabilized cycle is calculated using Eq. (2.2) and plotted in Fig. 7. Finally the remaining fatigue lives for critical location of both connections are calculated following the guidelines given in section 2.4. The obtained remaining fatigue lives for the all the critical connections are shown in Table. 4.

4.5 Member replacement/ strengthening scheme

The member replacement sequence and the time of replacement are then determined from the remaining fatigue life of each critical connections. These are shown in Table 5. The determined scheme were also compared with two previous strategies based member replacement schemes (similar schemes considered in section 3 in Table 1) as shown in Table 5. The comparisons revealed that the proposed strategy based scheme deviates from previous strategies based schemes.

Table 5. Comparisons of member replacement schemes

Member should be replaced (connection)	Previous strategy 1 (AASHTO, 1990)			Previous strategy 2 (Caglayan et al. 2009)			Proposed strategy		
	Rating factor	Replacement years	Sequence	Service life	Replacement years	Sequence	Service life	Replacement years	Sequence
TMB3	3.69	110	4	305	300	5	323	320	6
TMB5	2.70	50	3	156	150	3	165	160	4
TMB6	2.48	35	1	157	150	3	169	160	4
CG4-5	5.63	190	6	20	20	1	12	12	1
ST16-46	4.84	145	5	24	20	1	13	12	1
TDT1	3.39	110	4	179	170	4	45	45	3
TDT2	2.72	50	3	168	170	4	171	170	5
TDT3	2.61	40	2	131	130	2	24	20	2
TDT4	2.55	40	2	132	130	2	162	160	4

5. CONCLUSIONS

A remaining service life based maintenance strategy was proposed. The verification of the proposed strategy was conducted by comparing the predicted replacement scheme with sequence of experimental failure and fatigue life of selected test specimens. The proposed strategy was further utilized to determine the member replacement scheme of a riveted railway bridge. It was shown that the proposed strategy gives a much safest member replacement scheme for riveted railway bridges where detailed stress histories are known. Further verifications of proposed strategy are currently under way.

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