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Evaluation of Fatigue Remaining Life of Typical steel plate girder Bridge under Railway Loading

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Abstract: This paper presents various fatigue damage models available in the literature for calculating the damage index due to fatigue loading. Discussion on the effect of load level, load sequence and load interaction on the fatigue damage of the bridges is also presented. Experimental studies conducted to obtain the strain responses at the different critical sections of a typical steel plate girder bridge under railway loadings are briefly described. Strain time histories obtained at different locations including at the weld zone are used to evaluate the fatigue remaining life of the bridge considered in this study. Load spectrum is developed by considering different types of railway loadings and under different speeds. It has been found that in low stress-low cycle cases, which are very common in bridges, the damage indices obtained by using different models proposed by researchers are not in close agreement and sometimes, the results are unrealistic since the low stress variations obtained from the experiments need to be considered. Further, it is opined that an exclusive model is necessary for such cases which can facilitate to reasonably calculate the fatigue remaining life of bridges.

Keywords: Fatigue, Damage models, Strain histories, Remaining life.

1 Introduction

Fatigue and fracture are cumulative damage process and many structures including bridges collapsed due to the fatigue and fracture. According to the study carried out by the ASCE Committee (1982) on fatigue and fracture, approximately 80-90% of failures in steel structures are related to fatigue and fracture. So, it is necessary to evaluate the fatigue remaining service life of those structures. Fatigue damage increases with applied loading cycles in a cumulative manner which may lead to

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fracture. Many engineering structures including bridges are subjected to variable amplitude cyclic loads, and fatigue damage is one of the main forms of failure of those structures. Bridges are subjected to complicated pattern of loadings which consist of a large number of low stress cycles developed in bridge members. Due to loading over a period it initiates the fatigue cracks or accelerates the propagation of cracks inside bridge members. So, it is important to evaluate the fatigue damage accumulation under traffic loading. Further, many countries have revised their codes of practice to allow higher loading on the bridges. To assure adequate safety of the structures, it is immensely important to first evaluate the structural response like strain and deflections at the critical locations of a bridge under different loadings scenarios and then to assess the fatigue remaining service life of the bridge. There is an ongoing process to develop new and more reliable fatigue damage models to predict fatigue damage models.

Researchers like [Fisher (1984)], [Fatemi and Yang (1998)] and [Banjara et al. (2010)] discussed on different damage models and their implications. [Zhou (2006)] used strain method for fatigue life evaluation of some of the bridges in the United States based on fatigue strength (S-N) curves. [Li, Chan and Ko (2009)] applied field strain measurement upon linear fatigue damage law and performed fatigue analysis according to Continuous Damage Mechanics (CDM) on a bridge in Hong Kong. In the present study, a steel plate girder railway bridge has been considered for evaluating fatigue remaining life of the same. Before proceeding further for evaluating of fatigue remaining life of the typical bridge based on the data obtained from an experimental investigation, few of the relevant damaged models proposed in the literature are discussed in brief to provide ready references to the readers.

2 Discussion of damage models

There are many fatigue damage models proposed by researchers such as Miner Linear damage rule, Marco-Starkey Model, Henry Damage Model, Corten-Dolon Model, Marine theory, Double Linear Damage Rule [DLDR], Damage Curve Approach [DCA], Double Damage Curve Approach [DDCA], Stress controlled theory, Strain controlled theory, Subramanyan's knee point Approach, Hashin-Rotem Model, Chaboche Continuum Damage Mechanics theory [CDM], linear elastic fracture mechanics [LEFM], to name a few. Before proceeding further for evaluating of fatigue remaining life, the models are discussed in brief to provide ready references to the readers.

2.1 Linear Damage Rule (LDR)

[Miner (1945)] first represented linear damage concept in mathematical form as the LDR expressed by

$$D = \sum r_i = \sum \frac{n_i}{N_i} \tag{1}$$

In this method, the measure of damage is simply the cycle ratio, where n_i is the number of cycles corresponding to the i^{th} load level and N_i is the number of cycles to failure corresponding to the i^{th}load level. From Figure 1, failure is deemed to occur when D $=\sum r_i = 1$, as shown in Fig.1; r_i is the cycle ratio corresponding to the i^{th} load level. The main deficiencies with LDR are its load-level independence, load-sequence independence and lack of load-interaction accountability. To overcome these deficiencies, different nonlinear damage models have been proposed by researchers.

2.2 Marco and Starkey Model

[Marco and Starkey (1954)] proposed the first nonlinear load-dependent damage model as

$$D = \sum r_i^{x_i} \tag{2}$$

where, x_i is a variable quantity related to the i^{th} loading level and for the different values of stress level, the nature of the damage function is shown in Fig.2.

2.3 Corten-Dolon Model

The general form of [Corten-Dolon (1956)] theory includes both stress-dependence and interaction effects. It is assumed that the number of damage nuclei produced by the highest stress in a spectrum will affect the growth of damage of lower stress amplitudes.

$$D = \frac{n_1}{N_1} + \left(\frac{n_2}{N_1}\right)\left(\frac{\sigma_{2a}}{\sigma_{1a}}\right)^d + \left(\frac{n_3}{N_1}\right)\left(\frac{\sigma_{3a}}{\sigma_{1a}}\right)^d + \ldots \tag{3}$$

Where, d is the material constant which is equals to 6.57 Mansur (2005), n_1 is the number of cycles applied at σ_{1a} and n_2 is the number of cycles applied at σ_{2a} and so on. Also, $\sigma_{1a} > \sigma_{2a} > \ldots$ And so on.

2.4 Double Linear Damage Rule (DLDR)

[Manson et al. (1967)] proposed the two-stages, i.e., crack initiation and crack propagation, of fatigue damage process for constant amplitude loading as shown in

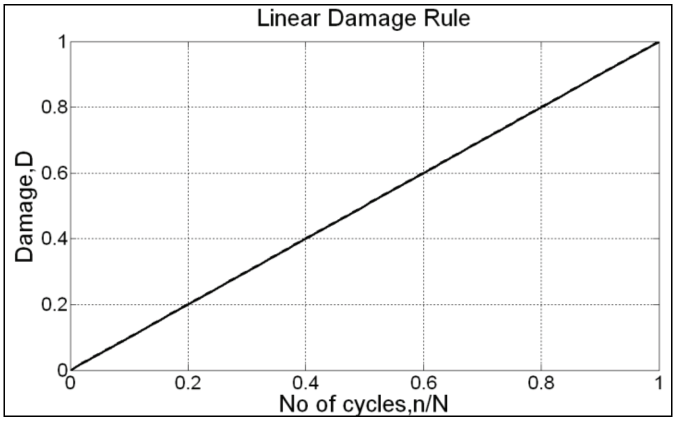


Figure 1: Linear damage rule (LDR)

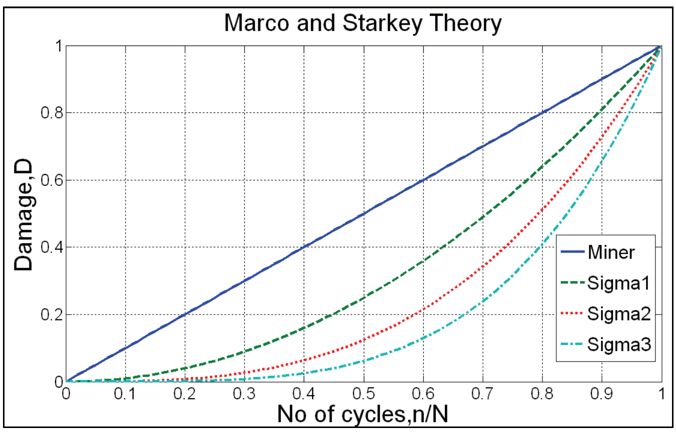


Figure 2: Marco-Starkey damage models for different stress (sigma) level

Fig.3. Instead of a single straight line, a set of two straight lines that converged at a common "knee point" would be used. The two stages were separated as given in Eq. 4 and Eq. 5.

For the crack initiation phase

$$\sum \frac{n}{N_0} = 1 \tag{4}$$

when,$N_f > 730$ cycles, $N_0 = N_f - 14N_f^{0.6}$ and when, $N_f < 730$ cycles, $N_0 \approx 0$.

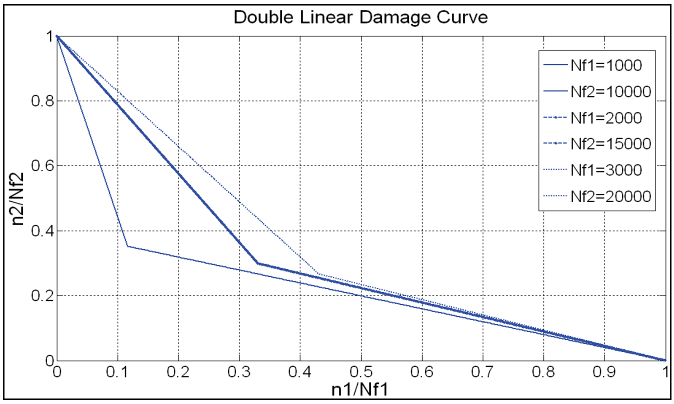


Figure 3: Double Linear Damage Curve Approach

For the crack propagation phase

$$\sum \frac{n}{(\Delta N)_f} = 1 \tag{5}$$

when,$N_f > 730$ cycles, $(\Delta N)_f = 14N_f^{0.6}$ and when $N_f < 730$ cycles, $(\Delta N)_f = N_f$.

2.5 Damage Curve Approach (DCA)

[Manson and Halford (1981)] empirically formulated the 'effective crack growth' model that accounts for the effects of crack growth processes, but without a specific identification. This model is represented by:

$$a = a_0 + (a_f - a_0)r^q \tag{6}$$

where,a_0, a and a_f are initial $(r = 0)$, instantaneous, and final $r = 1$ crack lengths respectively; and q is a function of N (maximum number of cycles of loading) equal to BN^β, where β is material constant. Damage (D) is then defined as the ratio of instantaneous to final crack length $(D = a/a_f)$. In most cases, $a_0 = 0$ and the damage function of the DCA simply becomes:

$$D = r^q \tag{7}$$

Fig.4 shows the typical variations of damage index in DCA with different N_r/N. If number of cycles N_r is selected, the constant B can then be expressed as $N_r^{-\beta}$. The exponent q in Eq. 6 can be written as $q = (N/N_r)^\beta$. Thus the factor q accounts for the load level dependence of the formulation.

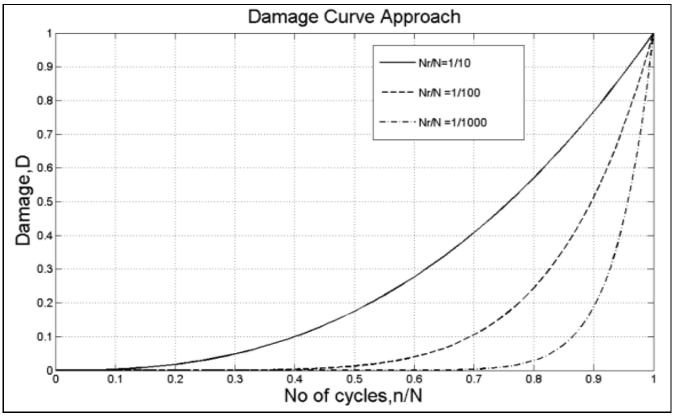


Figure 4: Damage curve approach

2.6 Double Damage Curve Approach (DDCA)

[Manson and Halford (1986)] improved the initially proposed model Manson and Halford (1981) by adding a linear term to the DCA equation with some mathematical manipulation, which is expressed as:

$$D = [(pr)^k + (1-p^k)r^{kq}]^{1/k} \tag{8}$$

where, 'k' is a mathematical exponent to give a close fit to the double linear damage line as shown in Fig.5, and 'p' is a constant measured from the slope of the first damage accumulation line in DLDR:

$$p = \frac{D_{knee}}{r_{knee}} = \frac{A(\frac{N_r}{N})^\alpha}{(1-(1-A)(\frac{N_r}{N})^\alpha)} \tag{9}$$

where, 'A' and α are the two constants and n_1 is the reference number of cycles selected.

2.7 LEFM Approach

In LEFM Approach, fatigue life is evaluated using the crack growth relationship [Paris (1960)].

$$\frac{da}{dN} = C(\Delta K)^m \tag{10}$$

where, da/dN is the crack growth rate in mm/cycles; 'C' and 'm' are the constants, which depend on material variables, environment, load frequency, temperature and

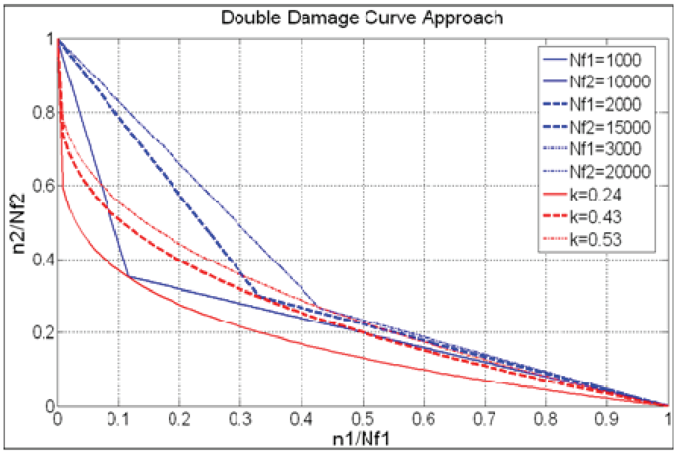


Figure 5: Double Damage Curve Approach

the applied stress range; ΔK is the stress intensity factor range which depends on the properties of the material, the crack orientation, size and shape of the crack, which is evaluated as

$$\Delta K = K_{\max} - K_{\min} = F(a,y)\sigma\sqrt{\pi a} \tag{11}$$

In which σ is the stress range; $F(a,y)$ is the geometry function to account for the possible stress concentration; 'a' is half crack size and 'y' is the vector of geometrical parameters. In case of an assumed semielliptical crack propagating at the toe of a weld on steel plate [Tsiates (1999)], the geometry function $F(a,y)$ is expressed as

$$F(a,y) = F_e F_s F_w F_g \tag{12}$$

where, F_e is the correction factor for crack shape, which is expressed as:

$$F_e = \frac{1}{\int\limits_0^{\pi/2} \sqrt{(1-(\frac{c(a)^2-a^2}{c(a)^2})\sin^2(\theta))d\theta}} \tag{13}$$

Based on [Fisher (1984)], cracks can be modeled as semi-elliptical surface cracks with the depth 'a' and length 'c' in the flange. So, 'a' and 'c' (in mm) have the empirical relationship as $c = 3.549a^{1.133}$.

F_s is the correction factor for the effect of free surface, and evaluated as

$$F_s = 1.211 - 0.186\sqrt{\frac{a}{c(a)}} \tag{14}$$

F_w is the correction factor for finite thickness of flange, and determined as

$$F_w = \sqrt{\sec(\frac{\pi a}{2t_f})} \tag{15}$$

where t_f is the thickness of the flange.

F_g is the correction factor for non-uniform stress acting on the crack, and calculated as

$$F_g = \frac{-3.539 \ln \frac{z}{t_f} + 1.981 \ln \frac{t_{cp}}{t_f} + 5.798}{1 + 6.789(\frac{a}{t_f})^{0.4348}} \tag{16}$$

where z is the weld size and t_{cp} is the cover plate thickness.

By using the technique of separable equation from differential equations and including the above mention factors (F_e,F_s,F_w and F_g) in the fatigue life in cycles, N can be found from Eq. 10 as

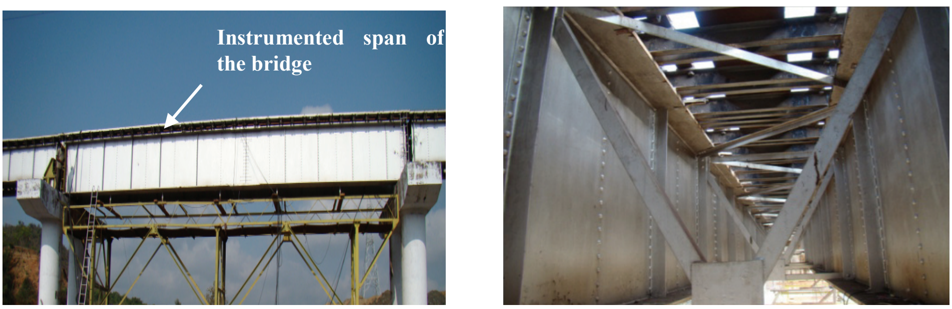
$$N = \frac{1}{A} \int_{a_i}^{a_{cr}} \frac{1}{(F_e F_s F_w F_g \sigma \sqrt{(\pi a)^m})} da \tag{17}$$

where, a_i and a_{cr} are the initial and critical crack depth respectively.

Since many fatigue models are already available, which can be applied for determining fatigue remaining life of bridges, it is utmost important to evaluate the efficacy of these models. One of the most important parameters for determining the damage index and the fatigue remaining life of a particular bridge is the number of cycles applied at different stress ranges. Hence, for calculating damage index, actual measurement of stresses during movement of the traffic over the bridge needs to be carried out. Further, to determining the remaining fatigue life of the bridge, the obtained structural response data should contain the representative traffic spectrum over the bridge. Therefore, in the present study, a typical railway steel bridge has been considered for experimental evaluation and a test train formation was run over the bridge at different speeds. The test train formation represents the proposed increased axle load for Indian Railway. Further, the structural response corresponds to scheduled passenger and goods trains, which represent the existing traffic load in that particular route, was also obtained. Experimental investigation on development of strains at different location of the bridge due to the movement of train has been carried out. A brief description on the bridge, instrumentation, test train details with load sequences and strain response obtained from the test are presented below.

3 Experimental Investigation

The bridge considered in the present study is a steel plate girder type bridge in AJJ – RU line and it comprises of one long span of 19.4m and two identical short spans 12.2m. Overall depth of the girder is 1.84m. The longitudinal and cross section view of the steel girder bridge is shown in Fig.6. The thickness of the flange and the web is 0.025m and 0.01m, respectively. To measure the structural response parameters of the bridge deck, data acquisition systems, computers (PCs), electrical resistance strain gages (both linear & rosettes) and LVDTs were employed. Strain gages were pasted at the selected locations on the top and bottom web along the length of the plate girder bridge as shown in Figs.7 and 8.



(a) (b)

Figure 6: Typical arrangement of on the bridge girder (a) longitudinal view (b) Cross Section of bridge

3.1 Test Train

In this study, a combination of different trains like goods train, express trains and passenger trains with different speeds have been considered. These trains represent a considerable variation in load distribution, number of wagons and locos, and speeds. Further, a test train formation was used for evaluating the structural responses where the test train represents the proposed increase in loadings in the goods train in dedicated iron ore route. The test train formation consisted of two front locos of WAG7, 59 BoxN wagons with loaded iron ore, a BV-cabin and a rear loco as shown in Fig.9. Further details on the test train can be found elsewhere [Srinivas et al. (2010)].

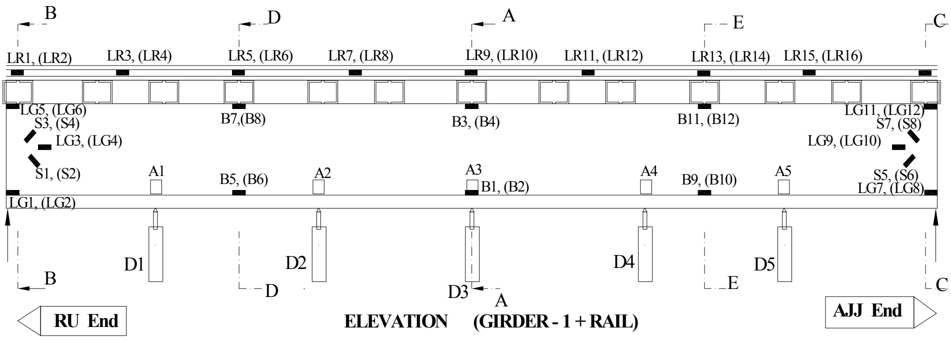


Figure 7: Instrumentation scheme for response measurement of steel plate girder bridge (LR represents strain gages on rail and B represents strain gages on plate girder)

Figure 8: Typical arrangement of strain gages on the flange and the web of the plate girder

3.2 *Experimental Results*

From the field test, strain time history during the passage of goods train, express trains and passenger trains with different speeds was recorded at the critical locations of the bridge. Strain gages were also pasted at the critical welded joints between web and flanges as shown in Fig.10. From the field test, strain time history at the welded section also has been recorded in the left and right side of the web as shown in Fig.11. The stress spectrum for different trains and different speeds at the welded section of the standard block of cycles is then obtained as shown in Figs.12 (a), (b), (c), (d), (e), (f), (g) and (h) and then combined stress spectrum is evaluates as shown in Fig.13.

Figure 9: Formation of test train over the steel bridge considered in the present study

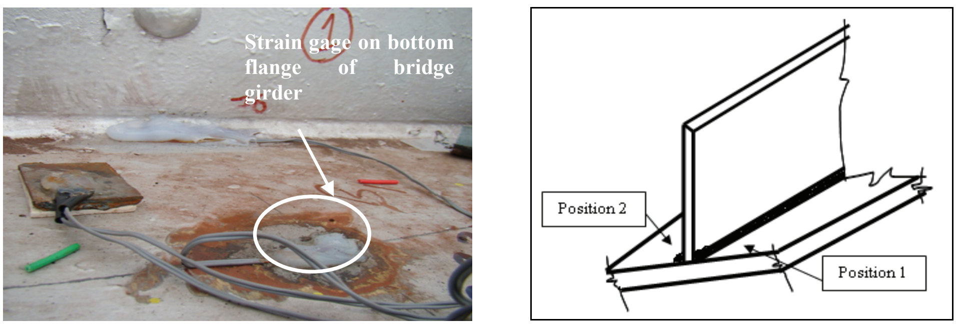


Figure 10: Locations of the strain gages at the welded section

4 Results and Discussion

Fatigue remaining life of the bridge was evaluated using the damage models discussed in the preceding section. It is assumed that every day on an average 21 goods train, express trains and passenger trains with different speed and different loadings are passing over the bridge. In this study, three cases have been considered for calculating the damage index by different damage models. (i) The first case considers all the recorded stress cycles without filtering low stress cycles, (ii) in the second case the number of stress cycles between 0-5 MPa were ignored and (iii) in the third case, number of stress cycles exceed one third of endurance limit stress were considered which means the factor of safety is considered as three. This quantification of factor of safety is arguable and may vary from bridge to bridge. It is easy to understand that the factor of safety has great influence in evaluation of remaining service life. Damage indices calculated using the strain data obtained

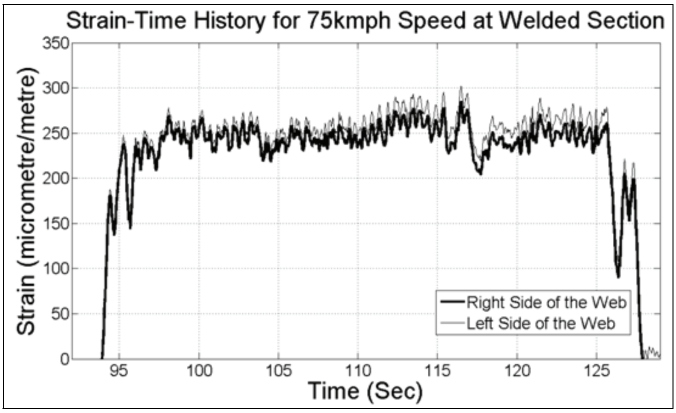


Figure 11: Strain-time history on weld section of flange plate

from the experiment are given in Table 1.

Table 1: Damage index for different damage models

Damage Models	Damage (with noise)	Damage (without noise)	Damage (endurane/3)
Miner	5.56	1.76	0.10
Marco - Starkey	4.59	2.60	0.032
Corten - Dolon	4.46	1.47	0.087
Damage Curve Approah	7.5	1.71	0.043
Double Damage Curve Approah	6.85	1.56	0.05
Double Linear Damage Rule	5.81	1.85	0.12

From Table 1, it is observed that damage indices obtained from different models depend on different parameters such as material constants, range of stress applied, endurance limit etc. Damage index calculated using Miner law, Marco-Starkey, Corten – Dolon, DCA, DDCA and DLDR are found to be more than one in the first two cases but in the third case damage index is well below one. When all the cycles irrespective of their stress range are considered, it is found that the total number of cycles already imposed (till 2011) on the bridge exceed the number of cycles corresponding to failure of the bridge due to fatigue. Therefore, it shows the damage index more than one is not realistic. Even though all models except Miner rule are nonlinear models, while calculating the remaining service life (damage index equal to 1), the power of ratio (n/N) will not the affect on service life. Therefore, the damage models, Miner, Marco-Starkey, Corten-Dolon, DCA, DDCA and DLDR

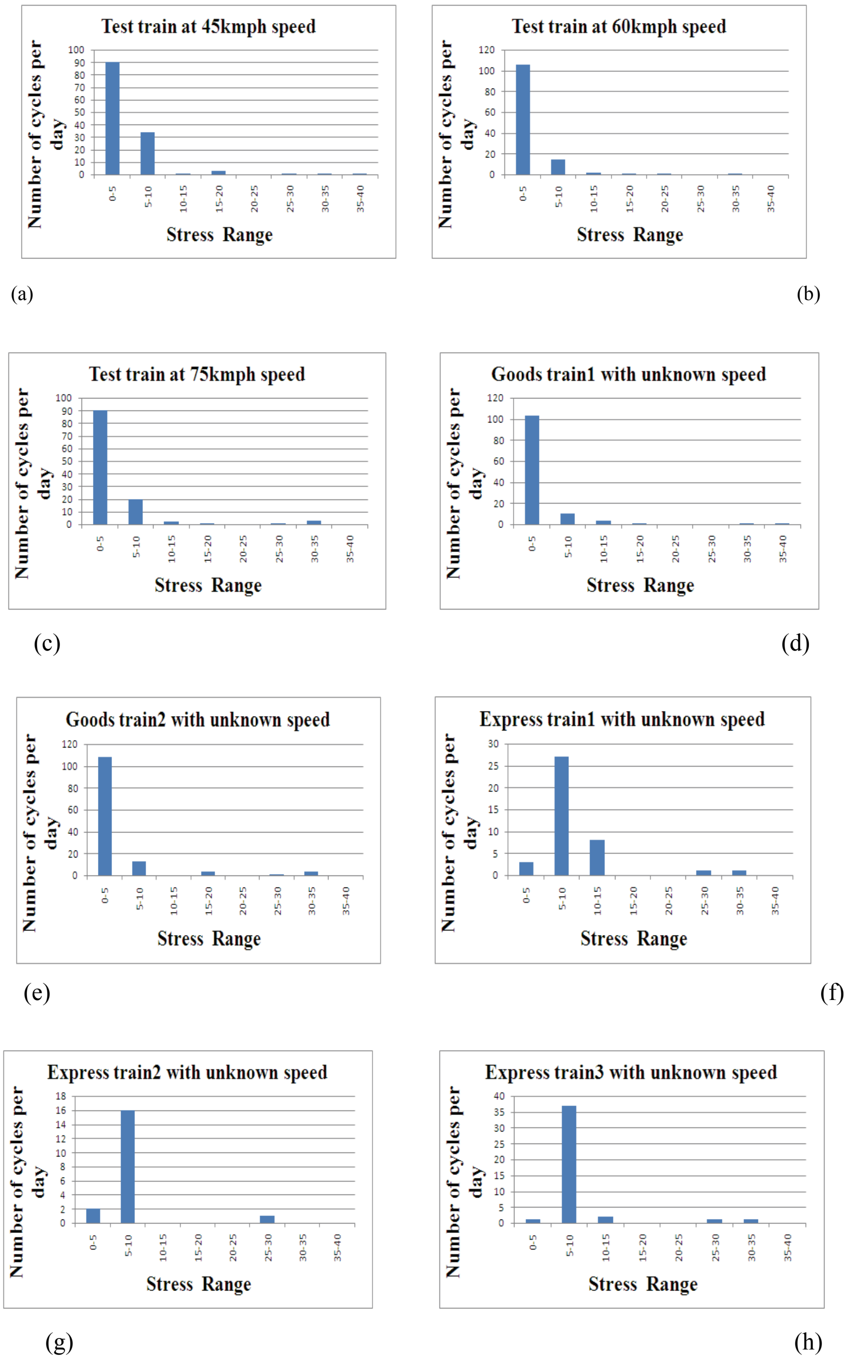


Figure 12: Stress spectrum at different speeds: (a) test train at 45kmph (b) test train at 60kmph (c) test train at 75kmph, (d) goods train1 (e) goods train2 (f) express train1 (g) express train2 (h) express train3

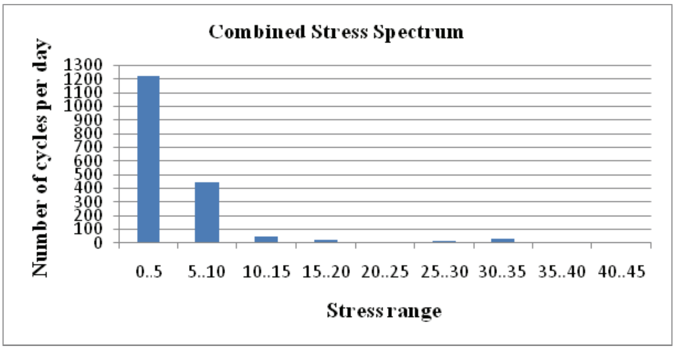


Figure 13: Combined stress spectrum

gives the fatigue remaining service life of the bridge as 149 years. In LEFM approach by considering initial and final crack length for the same bridge are 0.5mm and 1.5mm, the remaining service life of the bridge is found to be 167 years.

5 Conclusions

In this paper, assessment of fatigue remaining service life of a steel bridge has been discussed. Different fatigue damage models proposed by different researchers have been discussed first. Further, the models discussed in the present study are used to evaluate the remaining service life of a steel bridge in Indian Railway using the strain-time history responses obtained from the experimental investigations. The entire stress spectrum is discretized in 5 MPa bands. It is also important to mention that during experimental stress measurements, each train would provide large number of cyclic loading with very low stress variations which would lead to an unrealistic record of fatigue cycles. For considering the number of cycles under a particular stress range, three different considerations (i.e. with noise, without noise and one-third endurance stress limit) are made and the damage indices using different damage models have been evaluated. It is envisaged that the unwanted cycles need to be judiciously excluded for calculation of remaining service life of a bridge. It is observed that the results obtained from different models, where applicable, are not in close agreements. Further, it is opined that an exclusive fatigue model for low stress fatigue problem needs to be developed to evaluate the remaining service life of the bridges.

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