SECM/15/051

Life evaluation of critical members of steel bridges located in different atmospheres

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Abstract: Most of the iron and steel bridges in Sri Lanka are more than 100 years old. Since many of them are reaching or exceeding their design lives, the risk of collapsing those bridges have increased. One of the probable damages which bridges experience due to increasing traffic volume as well as environmental degradations such as corrosion is the corrosion fatigue failure. This problem is severe in bridges located in industrial areas and along the coastal line of the country. Corrosion and corrosion fatigue made a huge attention in the recent past due to many failures of bridges all over the world.

This paper presents a study conducted on assessing the corrosion fatigue damage of steel bridges. It proposes a procedure developed using existing fatigue and corrosion models for evaluating the remaining fatigue life. The procedure includes condition surveys, field loading tests, finite element modelling and analysis, developing S-N curves for different atmospheric conditions, use of corrosion rates and assessing cumulative fatigue damage. The paper also presents a case study: a century old iron and steel (mild steel and wrought iron) railway truss bridge damaged by both corrosion and fatigue. Using details of condition survey, load testing, appropriate corrosion data, finite element modelling and a corrosion fatigue assessment procedure, the remaining life of the bridge was evaluated for two atmospheric conditions; (i) corrosive atmosphere and (ii) noncorrosive atmosphere. The results of the evaluation were then compared to show the impact of the atmospheric condition on the fatigue life of the bridge.

Keywords: metal fatigue, corrosion, steel bridge, damage assessment, life evaluation

1. Introduction

Sri Lanka has large number of railway and roadway steel and iron bridges that are rapidly aging and are in continuous need for regular inspection, monitoring and maintenance. Damage assessment plays a vital role in this. One of the main components of an effective structural damage assessment is determining the remaining fatigue life of the structure.

Corrosion is considered to be an influential factor that significantly contributes to the reduction of the fatigue life. Corrosion process is highly accelerated in marine-industrial regions where chlorides and sulphates are regularly available [1,2]. Laboratory fatigue testing is the most accurate method for determining the fatigue life of materials. The drawbacks are that such tests are costly, time consuming and interpreting lab test data for real structures is difficult [3].

The "stress-life (S-N)" method is the most used fatigue damage assessment method [4]. This method uses the alternating stress amplitude (σ_a) to predict the number of cycles to failure (N). The S-N curve (stress amplitude versus fatigue life curve) that comprises the influence of material, geometry and surface condition is the tool used in S-N method. The S-N curve for a material can be developed using experimental tests. However due to the difficulty of conducting experiments, empirical and analytical formulae are available for developing material specific S-N curves. The S-N method is generally used for high cycle fatigue (HCF) where the stresses are low and material response is mostly elastic that is observed in steel bridges subject to usual service loading [5].

The reduction of thickness of metal elements due to corrosion is another problem that needs the attention. It is known that the corrosion is mainly depends on the atmospheric conditions. ISO 9223 [6] classifies the atmospheric corrosivity using; (i) pollutants and the time of wetness (TOW) and (ii) corrosivity rates, and points out five corrosivity categories. These corrosivity categories (numbered from C1 to C5 from low to high corrosivity) represent rural, urban, industrial and marine environments [7]. There are various methods and models developed to predict the rate of reduction of thickness subject to atmospheric corrosion [8,9]. Some of these models are based on the quantities of pollutants in the atmosphere and TOW while some are based on measured rate of reduction of thickness in different atmospheric conditions. These methods and models are very helpful analytical / empirical tools for assessing the corrosion damage up to a reasonable accuracy.

Accurate corrosion fatigue damage assessment of a structure needs a procedure that is developed combining the findings of corrosion fatigue and atmospheric / environmental conditions. Present paper attempts to propose an accurate procedure for assessing the corrosion fatigue damage of existing steel structures.

2 Hypotheses

The main cause of failure due to corrosion fatigue is broadly divided into two: (i) Reduction in thickness that increases the stress in the remaining part of the material and (ii) Roughening of the surface and pitting that act as stress raisers due to which cracks initiate earlier and, weakening of the material due to micro-structural changes caused by the combined effects of corrosion and cyclic loading. Therefore, to assess the corrosion fatigue damage of a structure the damages of both these causes should be taken into account.

2.1 Prediction of reduction of thickness

Corrosion rates of steels are largely different from one atmosphere to the other. Using corrosivity categories of ISO 9223 [6] with different atmospheres, the corrosion rates of carbon steel for the first year of exposure are in the range, rural: 0.0 - 1.3 μm/a, urban: 1.3 - 25 μm/a, marine: 25 - 80 μm/a and industrial: 80 - 200 μm/a [7]. Studies also show that the rate of corrosion decreases with time. Four years experimental data for carbon steel shows that the percentages of the average corrosion rates at the 2nd and 4th years with respect to the 1st year are 63% and 42% respectively. Further, the study of corrosion rate of carbon steel in seawater shows that the average corrosion rate after the 5th and 20th year are 70 and 50 μ/m [10,11]. Akesson [12] has mentioned that the corrosion rates of riveted bridges (wrought iron and mild steel) located in industrial areas is about 50 μm/a and in the inland part of the country (Switzerland) is less than 5 μm/a. Pitting (localized corrosion) is usually observed in structures exposed to the atmosphere due to the effects of the non-uniformity of material, environmental and corrosivity conditions [1,10]. The pitting factor is the ratio between the deepest metal penetration and the average metal penetration due to corrosion. Studies show that the pitting factor for steels (without mill-scale) is in the range 2 – 3 [10]. Accordingly, four methods can be suggested to obtain the rate of thickness reduction; (i) measuring the thickness reduction rate experimentally, (ii) measuring the pollutant levels and TOW first and then determining the rate using ISO 9223 or any other empirical methods, (iii) calculating an average rate by measuring the actual thickness of existing structures in which original thickness is known and (iv) using available data of previous studies. In the present study, the method (iii) was used to obtain the average thickness reduction rate of the structure.

2.2 S-N curve for corrosion fatigue

Mathematical functions have played a major role in developing S-N curves. In this study, a simplified corrosion fatigue S-N model that was developed using Palmgren function was used as the basic tool for the fatigue life evaluation [13,14,15].

The well accepted fact on the impact of corrosion on the fatigue damage is that corrosion reduces the fatigue life of metallic materials [12,16,17]. Therefore, in the S-N approach based fatigue assessment, the effect of corrosion should be included in the S-N curve. Accordingly, the corrosion based S-N model [17] used in the present study is given in Eq. (1),

$$\sigma(N)_{corr} = \sigma(N) - d \cdot log(N) \qquad (1)$$

where, $d = \frac{(\sigma_{VHCF} - \sigma_{corr})}{log(N_{VHCF})}$., $\sigma(N)$ and $\sigma(N)_{corr}$ are the stress amplitudes required for the fatigue failure of the element at N cycles in non-corrosive and corrosive atmospheres respectively, N_{VHCF} is the number of cycles concerned in the very high cycle fatigue (VHCF) region (> 10^7 cycles) and σ_{VHCF} is the fatigue strength amplitude in the VHCF region.

2.3 Stress range and mean stress correction

The S-N curve obtained in Eq. (1) is the standard curve for constant amplitude zero mean stress condition (stress ratio $R = -1$). However, in bridges, the stress amplitudes are neither constant amplitude nor zero mean stress. Therefore, stress ranges are calculated using the reservoir (or rein-flow) counting method and mean stress correction is carried out using modified Goodman method [17, 18] given in Eq. (2),

$$\sigma_a = \sigma_{ar}\left(1 - \frac{\sigma_m}{\sigma_u}\right) \qquad (2)$$

where, σ_{ar} is the corrected stress amplitude, σ_a is the stress amplitude, σ_m is the mean stress and σ_u is the ultimate tensile strength.

2.4 Damage accumulation method

Stress ranges applied on a bridge varies during its service life (variable amplitude loading). The objective of using a damage accumulation method is to obtain the cumulative damage caused by each and every stress range. Though there are many methods available for estimating the cumulative fatigue damage, due to its simplicity, the most used Palmgren - Miner rule [4] was used in the present study. According to Palmgren - Miner rule, the cumulative damage D_i caused by n_i numbers of cycles of stress amplitude σ_i is given in Eq. (3),

$$\sum_1^n D_i = \sum_1^n \frac{n_i}{N_i} \qquad (3)$$

where, N_i is the number of cycles to failure at the stress amplitude σ_i. The failure occurs when $\sum_1^n D_i \geq 1$.

2.5 Damage assessment procedure

Combining methods and techniques described in sections 2.1 – 2.4, the procedure proposed for assessing the cumulative fatigue damage of a steel structure in non-corrosive and corrosive atmospheres is given below (Figures 1 and 2).

Step 1: Measure dimensions of the bridge on-site, conduct a condition survey and field loading tests; Model the bridge (with measured thicknesses) using finite element methods (FEM), validate the model using measurements obtained from field loading test; Identify the critical members of the bridge based on stress levels and cycles; Prepare detailed FEMs of these critical members to obtain stress concentration factors SCF.

Step 2: Determine past and future loading details of the bridge (for railway bridges using railway time tables and for road bridges using traffic survey data). This detail should include the type of the vehicle, weight, number of wheels and wheel spacing etc.

Step 3: Use the validated FEM to obtain the influence lines of each critical element for all vehicle types; Using these influence lines, determine the primary force ranges (reservoir counting) and number of cycles n_i for these critical members.

Step 4a: For structures with no corrosion: Calculate the primary stress ranges ($\Delta\sigma_{p,i}$) using force range and original thickness; Then calculate the effective stress ranges at critical location (using SCF) with mean stress correction $\Delta\sigma_{e,i}$; Prepare histograms of $\Delta\sigma_{e,i}$ and n_i.

Step 4b: For structures with corrosion: Divide the past life and future life of the structures in to several intervals (say 10 year intervals); predict the actual thicknesses at each interval based on the corrosion rate; Calculate the primary stress ranges ($\Delta\sigma_{p,i}$) using force range and predicted thickness for all the time intervals; Calculate the effective stress ranges at critical location (using SCF) with mean stress correction $\Delta\sigma_{e,i}$; Prepare histograms of $\Delta\sigma_{e,i}$ and n_i.

Step 5a: For structures with no corrosion: Develop the material specific S-N curve of steels with no corrosion; Obtain N_i values for each and every $\Delta\sigma_{e,i}$.

Step 5b: For structures with corrosion: Develop the material specific S-N curve of steels with corrosion; Obtain N_i values for each and every $\Delta\sigma_{e,i}$.

Step 5: Use Palmgren – Miner damage accumulation rule or any other method to calculate the cumulative fatigue damage and the future life.

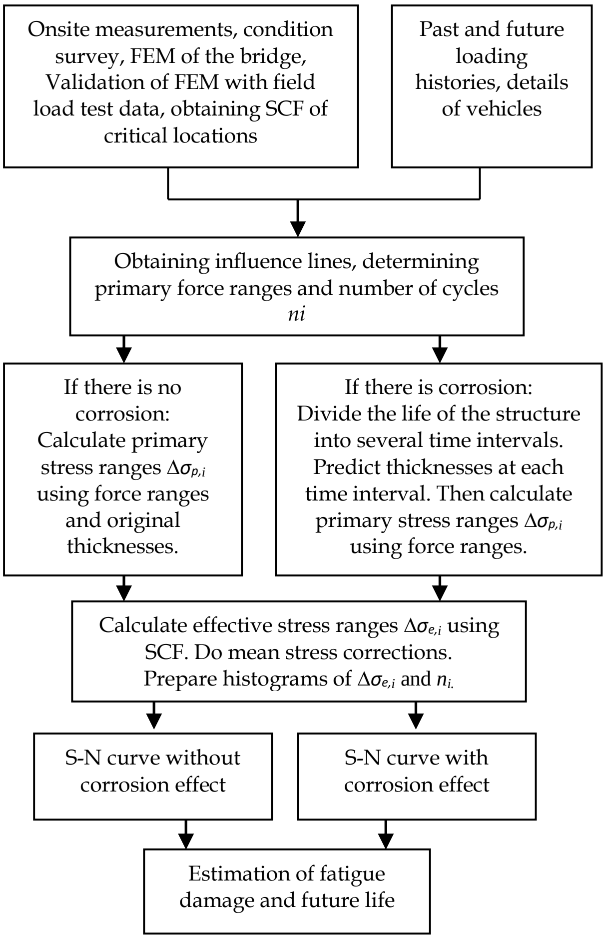


Figure 1: Damage assessment procedure

3. Case study

A century old iron and steel (mild steel and wrought iron) railway truss bridge damaged by both corrosion and fatigue was used for the case study. The bridge is a semi through truss Girder Bridge (Figure 2). The span and height of a truss is 32.25 m and 3.20 m respectively. The bridge is 5.35 m wide and caries a single rail track of gauge 1.74 m. Century old trusses of the bridge are wrought iron while some of the cross girders that were changed in 1980s are mild steel.

Figure 2: A view of the steel truss bridge

3.1 Onsite measurements

Condition survey and field loading test were first carried out. Two engines of mass 66 tons each were used for field loading test. Static loading was done by placing the engine on the bridge at several positions while dynamic loading was done when the engines are moving with different speeds. Electronic data measurements were obtained using strain gauges, acceleration gauges and displacement gauges.

In order to obtain the properties of materials used in the bridge, onsite non-destructive hardness testing was carried out using a portable tester. Later, material samples received from the bridge owner were tested at the laboratory. Table 1 gives the mechanical properties of wrought iron and mild steel.

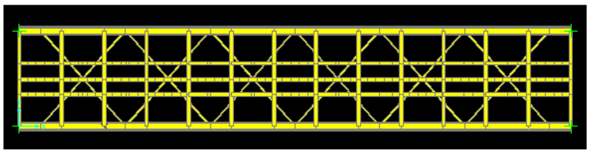
Table 1: Mechanical properties of steels

Steel	σ_u (N/mm^2)	*Hv* (kgf/mm^2)	σ_y (N/mm^2)	*El* %
Wrought iron (WI)	343	104	230	18
Mild steel (MS)	426	120	249	37

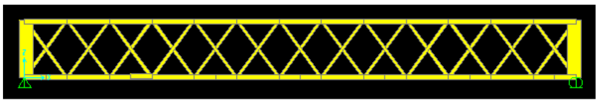
(*Hv* is the Vickers hardness, σ_y is the yield strength and *El%* is the percentage elongation at brake)

3.2 Finite element models

Based on the detailed structural drawings prepared from field measurements, a three dimensional FEM for the whole bridge (Figure 3) was developed. Material properties obtained from laboratory testing were used in FEMs as required. The elastic modulus used was 205 GPa and Poisson's ratio used was 0.3. Loading test measurements (deflection, strain and acceleration) were used to validate the FEM. Suitable adjustments at support and joints were carried out to improve the performance of the model. Few detailed FEMs were also developed for critical members in order to observe the stress pattern and to obtain stress concentration factors (Figure 4).



Plan viwe of the bridge showing bottom chord, cross girders, stringres, bottom bracing and the rail track.



Sectionl viwe showing dioganal members, top and bottom chord and end box.

Figure 3: FEM of the bridge

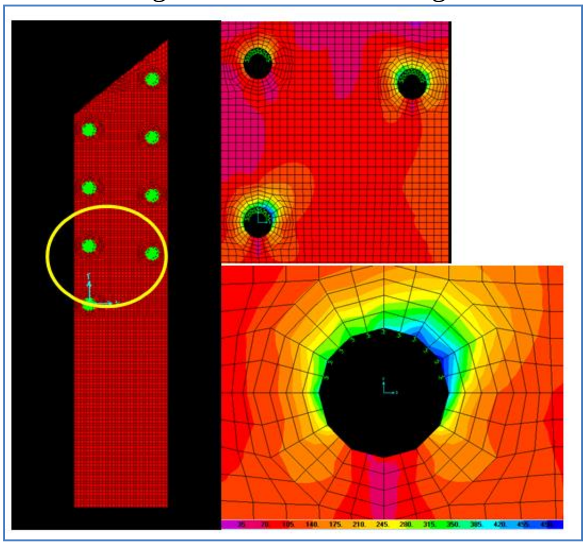


Figure 4: FEM of a critical connection

3.2 Full range S-N curves

Full range S-N curves for wrought iron and mild steel were developed for corrosive and non-corrosive conditions using the S-N models described in Section 2. As the S-N models produce the mean curves which has a 50% probability of failure (probability of survival Ps = 50%), 90% probability of survival (Ps = 90%) curves were developed for fatigue damage assessments [13,17]. The uniform scatter of stress (Tσ) for metals was estimated by using Eq. (4),

$$T\sigma = 1 : [\Delta\sigma(Ps=10\%)/\Delta\sigma(Ps=90\%)] \qquad (4)$$

Figures 5 and 6 show the S-N curves for wrought iron and mild steel developed for non-corrosive and corrosive atmospheres.

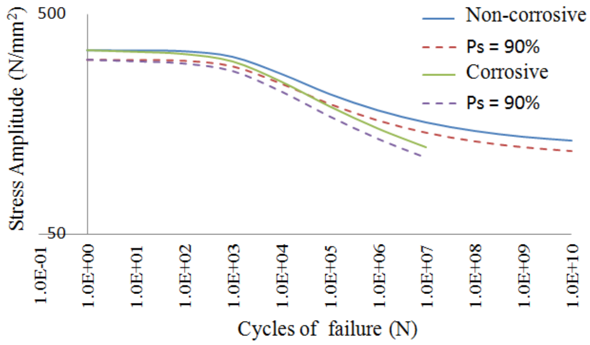


Figure 5: S-N curve for wrought iron

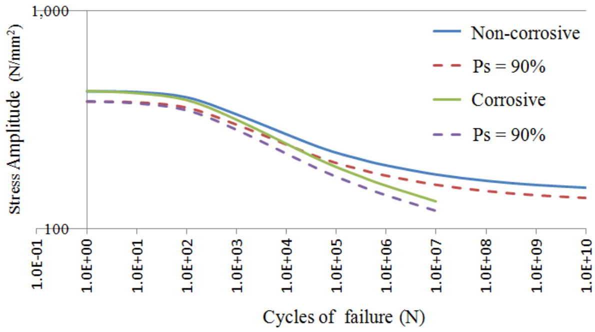


Figure 6: S-N curve for mild steel

3.3 Cumulative fatigue damage assessment

As described in Section 2.5 the critical members of the bridge were identified from the validated FEM of the bridge. The detailed FEMs of these critical members were then developed in order to obtain the stress concentration factors. Train timetables were used to obtain the past and future loading of the bridge. Thickness reduction due to corrosion was found by onsite thickness measurements and then the thickness reduction rate was estimated. Using influence lines of forces in critical elements and stress concentration factors obtained from FEMs, and modified Goodman method the effective stress ranges were calculated. Then, using the S-N curves developed, the fatigue damage with and without corrosion effect was estimated. Palmgren – Miner damage accumulation rule was used to find the cumulative fatigue damage.

3.4 Results

When considering the cumulative fatigue damage of one of the critical members: diagonal member 1, the total life for non-corrosive atmosphere was 180 years whereas that in corrosive atmosphere was 60 years. In the actual structure, some of the diagonal members are in damaged state. Some of the members have been replaced using mild steel, i.e., main girders. Therefore the wrought iron members may not survive up to 180 years. However, there were no observable fatigue cracks in the diagonal member 1 to verify the life of 60 years. One possibility is that the members may have deformed (failed) and the loading may have been distributed among other diagonal members. The other possibility is that the predictions of the procedure proposed may be conservative. Therefore, further research may be required to verify or improve the procedure proposed.

4. Discussion and conclusions

Corrosion fatigue damage assessment of steel structures needs research as there are no successful methods available. In the present study, an attempt was made to propose an assessment procedure that comprised different validated techniques. The advantage of using the procedure proposed is it uses a material specific S-N curve that represents the material behaviour and corrosion rates that represents the atmospheric conditions. The separation of (i) reduction in thickness that increases the stress in the remaining part of the material and (ii) micro-structural and surface changes of the material weakening the fatigue strength has made the proposed model simple. Even the method used for developing S-N curves for corrosive atmospheric conditions does not require expensive fatigue testing that makes the procedure simpler.

The conclusions of the study are as follows.

Existing theories and assessment methods can be used together with FEMs to assess the corrosion fatigue damage.

The fatigue damage of structural elements in corrosive atmosphere is considerably higher than the damage when they are in non-corrosive atmosphere.

FEM shows that the stress concentration factor of riveted connections increase with the reduction of the plate thickness due to corrosion.

Case study conducted using an iron and steel bridge showed that the procedure proposed is reasonable than the usual method. However, it may produce conservative results. Therefore, further studies are needed to verify or improve the procedure.

Acknowledgement

Department of Civil Engineering, University of Peradeniya, the Open University, Nawala and Ceylon Government Railway (CGR), Colombo.

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