FUTURE LIFE EVALUATION OF EXISTING STEEL BRIDGES

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Abstract

Bridges play a major role in road networks. Continuous health monitoring and proper maintenance of bridges is required to obtain the maximum service from them. Life evaluation of bridges is part of the health monitoring work, which is usually done when bridges get to the end of their design lives under designed service loading or after accidental situations such as overloading or partial collapse of bridges due to floods, landslide, cyclones, earthquakes, bomb blasts, traffic accidents or after unexpected traffic overloading.

Health monitoring and assessing structural soundness of existing bridges is important for making economical decisions on repairing, retrofitting or replacing them. Knowing the future life of existing bridges is important for planning new bridges before the old bridges collapse.

This paper describes various methods used for fatigue life evaluation of existing steel bridges, the use of stress-life & strain-life curves and finite element models. It explains the four types of fatigue which occur in structural elements due to different magnitudes of cyclic stresses. It also describes the importance of selecting the appropriate life evaluation method. A case study is given as an example in which some of these techniques were used.

Keywords: Health Monitoring, Fatigue Life Evaluation, Finite Element Models, Sustainability, Risk Minimization

1. Introduction

Bridges are an essential part of road and railway infrastructure. There are thousands of steel bridges all over the world and many more are being constructed. Construction is the first of the three stages involved in the life of a bridge. The second stage is the service stage where the bridge provides its service. This stage requires proper maintenance of the bridge and continuous monitoring of its health. The last stage is the demolition at the end of its lifetime. To get the maximum output from bridges as well as to determine the correct timing for their demolition / removal, correct health monitoring and life evaluation techniques are required.

There are various life evaluation methods for steel bridges. These have been developed for bridges with known loading schedules (such as railway bridges for which the train timetables are available and can be used as loading schedules, Siriwardane et al., (2007)) and are extended to approximate irregular loading situations (such as highway bridges where exact details for the vehicles travelling on it are not available, BS 5400:Part10). However, the speed of vehicles, the maintenance procedure etc., differ from country to country and therefore the techniques must be tested and modified for local conditions of the country.

The theories behind the health monitoring and life evaluation of steel bridges are mainly related to the theories of fatigue and fracture mechanics. Depending on the magnitude of cyclic stresses, there are four types of fatigue failure known as very high cyclic fatigue (VHCF), high cycle fatigue (HCF), low cycle fatigue (LCF) and ultra low cycle fatigue (ULCF).

The HCF is defined as fatigue due to stress below the general yield stress. The number of cycles for failure in HCF region is within 10^3~10^4 cycles to 10^7~10^8 cycles and above this the VHCF region starts. Most of the cyclic stresses due to service loading in bridge elements are within the HCF and VHCF regions. The LCF is defined as fatigue due to stress above the general yield stress and the number of cycles for failure is between HCF (less than 10^3~10^4 cycles) and ULCF. Such stresses (related to LCF) may develop in bridge elements due to overloading and minor accidents. The ULCF is defined as the failure due to fatigue in less than 10~20 cycles where the stress is above the general yield stress and the failure is due to ductile fracture. Stresses of ULCF region may develop during earthquakes while the bridge is loaded and due to blast loading etc.

Effect of one, two or all of those four fatigue types may act together on real bridges. The fatigue life evaluation should be a combination of those relevant types. Therefore, investigating the actual loading history prior to selecting the life evaluation method is very important.

2. Determining the Fatigue Failure

The Fatigue life of structural elements is evaluated by two common approaches which are the fracture mechanic approach and the bulk (traditional) approach.

The fracture mechanic approach considers two stages of failure which are, the crack initiation stage and the crack propagation stage up to fracture. The subjects of formation of slip lines or slip bands due to monotonic or cyclic loading, crack initiation through these slip lines (in the plane of maximum shear stress range), growing of cracks (along the plane of maximum tensile stress range) with continuous monotonic or cyclic loading, combining of cracks forming few major cracks and the fracture when the length of the crack exceeds the critical crack length are thought-out with experiments. Then using fracture mechanics theories and experimental results, the fatigue life of structural elements could be determined. This approach is important for analysing the fatigue failures of LCF and ULCF regions.

The bulk (traditional) approach involves developing the S-N (stress vs life) or ε-N (strain vs life) curves by conducting tests using material samples/specimens. The geometry, shape and details of specimens and the test method (i.e. axial tensile test, rotating bending test etc) should be determined in accordance with the details of the real structural element which is the subject of the evaluation. Using specimens under different stresses, the number of cycles to failure at the considered stress range can be obtained and using a series of tests, the S-N curve for a particular material and specimen type (notched, un notched etc.), can be obtained. Normally in these tests, the crack initiation and propagation are not separately considered. The S-N curves so obtained are mostly used for fully reversed (zero mean stress) uni-axial stress of constant amplitude, but can be modified for non-zero mean stresses by using modified Goodman equation etc. S-N curves are widely used in the HCF region and ε-N curves in the LCF region.

VHCF and HCF regions are considered in the elastic range (in the macroscopic scale) due to low stresses in structural elements generated by cyclic loading. The LCF is related to high stresses in structural elements generated by cyclic loading. As the stresses are higher, there are plastic strains. Considerations of strain hardening and softening effects, ways of crack initiation and propagation and the concept of total strain which is the sum of the elastic strain and the plastic strain etc., are important in this LCF region. The ULCF is for a very few loading cycles with larger stresses. Therefore, for the analysis of ULCF, the theories and considerations used in LCF and deeper analysis on fracture mechanics are needed.

3. Fatigue Life Prediction

There are various life prediction methods available. The method given in BS 5400: Part 10: 1980 is one of the most widely used methods to evaluate the fatigue life in the HCF region. It uses an S-N curve with two standard deviations below the mean line with 2.3% probability of failure and the Palmgren-Miner rule to obtain the cumulative damage. The method developed by Siriwardane et al (2007), uses a full range S-N curve that is constructed using Kohout & Vechet technique and can be applied for both HCF and LCF regions. This method takes the effect of the loading sequence into account by using a damage indicator based sequential low described by Mesmeque et al (2005). Later, following Siriwardane's procedure, a combined ε-

N curve was developed by Karunananda et al (2010) for LCF and ULCF regions. There are some other HCF life evaluation methods such as AASHTO Guide Specifications for Fatigue Evaluation of Steel Bridges and the method in the report "District line fatigue of riveted under bridges, Infrastructure Consultancy Service (ICS), London underground limited. There are methods to develop ε-N curves for LCF using basic principles such as Manson – Coffin relationship that is developed using a combination of the elastic strain and the plastic strain.

3.1 Basic Method for Life Prediction

The S-N curve for smooth or notched elements can be built for the required failure probability with necessary adjustments for self stresses etc. Using the loading history of the structure, the cyclic loading (stress) history of the structural element concerned is numerically evaluated (i.e. using calculations or FEMs). Then those stresses are divided into several stress ranges and the number of cycles corresponding to these stress ranges is counted using the reservoir counting, the rain-flow counting or any other appropriate method. Then the damage; n/N, (n is the number of repetition of stress ranges that the structural element has been subjected to and N is the allowable number of repetition of stress cycles before failure) for each stress range is obtained from the S-N curve, neglecting the effect of minor wiggles. Addition of these damages is the total damage which should be less than 1. Using the damage so obtained and estimating expected future stress ranges, the number of future cycles before failure could be calculated and hence the expected future life is calculated.

3.2 BS 5400 - Part 10: 1980 – Clause 9.3 for Railway Bridges

The procedure described in BS 5400: Part 10: 1980, Clause 9.3, damage calculation for non standard loading (where past loading histories are not available) is one of the most widely used methods to evaluate the future fatigue life of railway bridges. Using table 17 of the Code, the detail class for welded or riveted (bolted) connections of the bridge could be first determined including the effect of the location for the potential fatigue crack initiation in a joint. The S-N relationship and non propagating stress range (σ_0) values are obtained from table 8 of the Code. If a considered stress range is entirely in the compression zone, effect of fatigue loading is ignored as per Clause 6.1.3. The Clause 11.3 is applied for low stress cycles. Then the ratio n/N is determined for all the stress ranges. The summation of n/N is obtained using Palmgren-Miner rule which gives the damage. Then using predicted future cyclic stress ranges, the future life of the structural element is calculated.

3.3 BS 5400 - Part 10: 1980 – Clause 8.4 for Highway Bridges

BS 5400: Part 10: 1980, Clause 8.4 can be used for the life evaluation of steel highway bridges. This method is applicable for any detail of bridge elements for which S-N relation is known and for any load or stress spectrum. The damage calculation of this method is also prepared based on Palmgren-Miner rule. The detail class of the connection or joint is determined using table 17, and the stresses and stress ranges at each detail class is calculated as per Clause 6 (Cl.11.4.b). Then the design spectrum is determined as per Clause 8.4 or 9.3. The procedure is described in Clauses 8.4.2 and 8.4.3. The Clause 11.3 is applied for low stress cycles. The damage calculation could be done as per Clause 8.4.4 which also refers to Clauses 11.2, 11.4 (c) and (d).

3.4 Sequential Low Based Method by Siriwardane et al.

Damage indicator based sequential law has been used in this method that captures the loading sequence effect of variable amplitude loading more precisely. The first step is constructing the full range S-N curve for the particular material. Then, for a considered stress range σ_i, having a number of cycles n_i, the damage indicator D_i can be obtained from $(\sigma_{i(eq)}-\sigma_i)/(\sigma_u-\sigma_i)$, where σ_u is the ultimate tensile strength amplitude (or range) for rotating bending test-based S-N curves and $\sigma_{i(eq)}$ is the i^{th} level damage stress amplitude obtained from the S-N curve for $(N_i – n_i)$ number of cycles. The damage indicator D_i is transformed to the next stress range σ_{i+1} having n_{i+1} number of cycles using the expression, $\{(\sigma_u-\sigma_{i+1})+\sigma_{i+1}\}/\sigma'_{(i+1)eq}$, where $\sigma'_{(i+1)eq}$ is the damage equivalent stress amplitude or stress range at the level i+1. The number of cycles $N'_{(i+1)R}$ that corresponds to $\sigma'_{(i+1)eq}$ is obtained from the S-N curve and the residual life at the level i+1, $N_{(i+1)R}$ is obtained from the expression, $N'_{(i+1)R} – n_{i+1}$. The stress amplitude $\sigma_{(i+1)eq}$ that corresponds $N_{(i+1)R}$ is obtained from the S-N curve. The Damage indicator of the $(i+1)^{th}$ level stress range D_{i+1} can then be determined from the expression, $(\sigma_{i+1(eq)}-\sigma_{i+1})/(\sigma_u-\sigma_{i+1})$. Applying the same procedure for all the stress levels, the damage indicator for the past loading can be obtained and then this indicator is used to calculate the future fatigue life for the expected loading.

3.5 Fracture Mechanics based Life Evaluation Models

In the case of LCF & ULCF, the life for crack initiation as well as the life for crack propagation is important. The crack initiation life is calculated using S-N or ε-N curves as explained in the previous sections. Fracture mechanics theories are applied for crack propagation stage. Fatigue crack growth rate (da/dN), stress intensity factor (K) and the range (ΔK), sigmoidal curve for the behaviour of da/dN vs ΔK, threshold value (ΔK^{th}) of ΔK (below which, there is no crack propagation), fracture toughness K_c (K at fracture = K_c), Paris equation (for the linear region, da/dN = A ΔK_n, where A is a constant and n is the slope of the sigmoidal curve), three basic crack extending modes (opening, shearing / sliding and tearing), use of the equation $K = S\sqrt{(\pi a)}\alpha$ (where S is the nominal tensile stress and α is a dimension less parameter), elastic stresses near a crack tip and elastic stress concentration factor (K_t), softening and hardening of materials due to cyclic stresses, theories for yielding, crack tip plastic zone and plastic zone size (r_y), notches and their effects, baseline (fatigue) notch factor (K_b), self stress and stress concentration at notches etc., are used in the analysis of fracture of material. There are various models developed using fracture mechanics theories (used in aerospace field) for variable cyclic loading such as the Wheeler model, the Willenborg model and the Elber model. Some of these principles could be easily applied with modifications for steel bridges.

3.6 Risk involved in the Life Evaluation

As the S-N or ε-N curves are developed for a certain probability of failure, there is a failure risk involved with all the methods that use S-N or ε-N curves. The methods developed with Palmgren-Miner rule do not sufficiently capture the effect of the loading sequence. Even though such methods predict reasonably accurate fatigue lives for HCF region, there could be errors for combined stresses of ULCF or LCF regions, Karunananda et al (2010). Further, as mentioned in the introduction, the combined effects of all four types of fatigue are not easy to determine. There could be missing loading conditions in the past loading history as well as unexpected loading conditions in the expected future loading which have not been counted in the life evaluation.

The case study given in chapter 4, table 1 shows estimated fatigue lives of 3 structural elements of a bridge evaluated using three different methods that shows different fatigue lives. This is an example which proves that there is a risk involved with the answers.

Therefore, selection of an appropriate life evaluation model, use of correct past loading histories, correct prediction of future loading conditions and using appropriate safety factors for the effects of corrosion and environmental interactions etc., are very important in order to minimize the risk of assessment related errors.

Regular inspections of critical structural elements, replacing those weak elements with stronger once (sufficient maintenance work) and onsite verification tests (field loading tests to measure strains and deflections) are some of the procedures which can be used to minimize unexpected failures of bridges.

4. The Case Study

The case study concerns a 40 years old, 34m long, 5.2m wide, single spanned, double lattice girded, wrought iron Railway Bridge, given in figure 1. The bridge is located at Puttalam (Bridge No. 02 on the railway track between the Puttalam Cement Factory and Limestone Quarry, used for limestone transport by trains) which was displaced from its abutments by floods several years before and re-erected on temporary timber abutments until it repair and retrofitting works in 2011. The main girders of the bridge are of equal span, each 33.65 m end to end, cross girders are placed at the panel points of the main girders, two longitudinal girders connected between cross girders in each panel and bracings between main girders.

The reason for conducting tests and doing health monitoring and life evaluations of this particular bridge was that there was a proposal to use locomotives heavier than the once used on the track and the authority wanted to know the structural soundness of the railway track and bridges to take such increased loading.

Figure 1 – Railway Bridge No. 2, Puttalam after repair & retrofitting.

4.1 Condition Assessment

A condition survey was first carried out which included a thorough inspection of individual structural elements, measuring thickness of structural elements, identifying corroded locations, missing elements and rivets etc. One of the important details found in the investigation was that the bridge was empty (not loaded) when the washing out of abutments and displacement of the bridge happened. Therefore, there were no cracks, damaged structural elements or any major defects observed during the inspection.

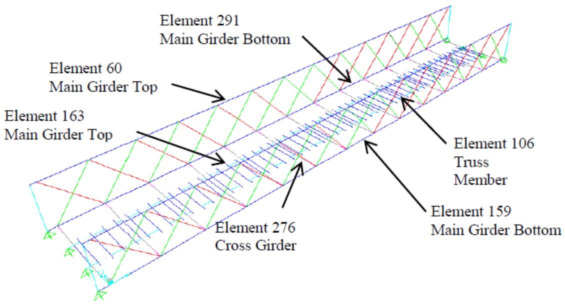

Figure 2 – The FEM and Critical Structural Elements

The analysis was done by means of a finite element model (FEM) of the bridge developed using SAP 2000 structural engineering software. Validation of the FEM was done by conducting loading tests on the bridge. Both static and dynamic loading tests were carried out by using a M2 locomotive with 6 numbers of 13.16 ton axles for 5 different loading cases to measure the displacement, strain and acceleration at pre-determined (critical) members of the bridge. Figure 2 shows the FEM and critical structural elements.

4.2 Structural Soundness & Future Life

Using the validated FEM, loading the FEM with expected higher loading (i.e. locomotive with 6 axels, 100 tons total mass) and obtaining stresses of critical structural elements, the ability of the bridge for higher loading situations was verified.

As there were no records for higher stresses in the structural elements (i.e. no LCF or ULCF) during the displacement of the bridge by floods, the fatigue life was calculated only using the stresses developed by locomotives (HCF) during its 40 years service (using train timetables). Cyclic stresses of critical elements were obtained from the FEM, loaded with static loading and multiplied by dynamic factors. The number of stress cycles of various stress ranges was obtained using the train time tables. Then, using the S-N curve for worth iron and considering detail classes at joints etc., the fatigue damage was estimated. The future fatigue life of the bridge was then estimated using expected timetable of trains with expected 100 ton locomotives. The future life so estimated is 30 years for critical members.

4.3 Retrofitting is More Sustainable

The health monitoring and life evaluation assessment showed that the structure is strong and has more than 30 year future life. Further, construction cost of a new bridge was estimated as much higher than retrofitting of existing bridge including new reinforced concrete abutments. Therefore, it was decided that rehabilitation of the bridge with necessary retrofitting works is more sustainable than constructing a new bridge. Accordingly the bridge was repaired, retrofitted and placed on new abutments.

4.4 Risk Minimization

Failure risk minimization of a bridge could be done by repetitive field loading tests. Analysing the fatigue life using various methods could help minimizing errors.

In order to verify the real behaviour of the bridge after its repair and to minimize the risk of errors of the life prediction, a field loading test and analysis was carried out. A M2 engine (i.e. locomotive with 6 numbers of 13.16 ton axles) was used for this test under different static and dynamic loading conditions and deflections and strains at critical structural elements were measured. As the verification tests have shown improvements with regard to deflections of the bridge and reasonable factors of safety for the critical structural elements with regard to strains and stresses, it was concluded that the repair and retrofitting was a success.

A comparison of future fatigue life of three critical structural elements calculated using three methods described in the previous sections (i.e. BS 5400: Part 10: 1980, ICS: London Underground Ltd., and Siriwardane et al.) are given in table 1.

Table 1 – Future Fatigue Lives of Critical Elements

Member	*Future Fatigue Life (Years)*		
	BS 5400	*ICS*	*Siriwardane et al*
Main girder	*1712*	*1404*	*1059*
Diagonal truss member	*443*	*277*	*94*
Middle cross girder	*408*	*182*	*93*

All three methods show different but better results; i.e. more than 30 years of fatigue life with a factor of safety of 3.

5. Conclusion

Most of the old highway steel bridges and railway bridges in Sri Lanka are nearing the end of their design lives. Knowing the future life spans of these bridges can help the relevant authorities to plan for their rehabilitation or replacement in advance, thereby contributing to sustainable development. The methods described in this paper can be applied practically in Sri Lanka to carry out such life assessments of existing steel bridges.

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