FATIGUE LIFE ESTIMATION OF STEEL TRUSS BRIDGES

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Abstract: Fatigue is the effect of alternating stresses acting for a long time over structural members. Such phenomenon is very common in bridges. In one passage of vehicle, generated stress may be very small but its cumulative effect will reduce the life span of the structure. Two approaches were used, first one is based on Miner's hypothesis of damage accumulation and second one is by linear fracture mechanics principle. The stress range histogram has been developed using Rain flow counting method. Number of cycles/year corresponding to each stress range has been calculated based on an assumed annual traffic flow over the bridge considered.

I. INTRODUCTION

Bridge structures are the main transportation systems that are adopted in the emerging world where it crosses a river, major channels, small islands etc. Bridges may be considered as the most vulnerable elements of the infrastructure as their condition of being out of service results in large economic losses and may create undesirable public opinion about the Government. All bridges deteriorate with increasing age and without timely corrective action, deterioration can leads to restriction on speeds, closing of the bridge, or in the worst case, catastrophic failure. In many countries, since highway bridges are designed for their design vehicles which are specified in the code, most of the engineers tend to believe that the bridge will have adequate capacity to handle the actual present traffic. This belief is generally true if the bridge was constructed and maintain as shown in the design plan.

II. TRUSS BRIDGE :

Truss bridges are very common in highways and road ways. It is economically used in the span range of 100 to 200 meters. A truss bridge is offering to measure structural advantages. The primary forces in the member are axial forces. Greater overall depth is permissible with its open way of construction. This will result in reduction of self-weight. The most common one is warren truss shown in fig 1.1. Various components of typical through type truss are floor beams, stringers, cross girders and lateral bracings provided to protect from wind action. The deck is of concrete is joined with stringers with the help of shear connectors. Stringers carry loads from the floors and transmits to the cross beams. The cross beam transfers the loads to joint of the truss. The lateral bracing consisting of struts and diagonals provides rigidity to the structure, stabilize the compression chord and carry the main part of the wind loads to the bridge portal. Secondary stress can also be called due to eccentricity in connections, torsional moments due to floor beams and truss distortion due to lateral loads. The joints of the truss members may be riveted or welded. Design of truss bridges require Type of the truss, Height of truss, the depth of the truss to length of the deck panel for the highway bridge can be taken as l/8 to l/20 where as it varies from l/5 to l/10 for railway bridge.

III. DYNAMIC EFFECT OF MOVING VEHICLES:

Bridge structures that have long service years or long spans, or that are frequently subjected to heavier loadings than their design loads, are greatly affected by heavy traffic induced vibrations. The most important parameters influencing the dynamic stresses in the bridge are: the frequency characteristics of bridge structures (i.e., the length, mass, and rigidity of individual members), the frequency characteristic of vehicles, weight of the vehicles, the damping in bridges and in vehicles, the velocity of vehicle movement, the track irregularities, and so on.

IV. FATIGUE ASSESSMENT DUE TO MOVING VEHICLES:

Sustainability is a key-issue for the design of bridges including steel bridges. The most important sustainability indicator for bridges is durability. Fatigue is an important consideration in the design of bridges, especially those made of steel. An increasing part of work on the roadway infrastructures concerns the assessment and maintenance of existing structures.

Fatigue evaluation is an important task in design of bridges because the metallic members of bridges are subjected to variable amplitude loading due to passage of traffic on bridge. For fatigue evaluation, stress ranges and number of cycles should be determined as accurately as possible. However, they are highly dependent of different parameters like bridge type, detail location, span length and vehicles axles' configuration.

In general, two different methods can be used in order to carry out a fatigue assessment, namely, the S-N method and the fracture mechanics method. The former, which is based on the S-N curve of the fatigue detail in question, is used in conjunction with Miner's rule (Miner 1945). By contrast, the latter method considers explicitly the growth of fatigue cracks and for this reason, it is more appropriate in cases where a fatigue crack has been detected. Since crack detection is mostly case-specific, most of the fatigue assessment methodologies that have been developed for railway bridges are based on the S-N approach.

3.1 MODELLIING OF TRUSS BRIDGE :

Two through type truss bridges with different configurations are considered for the analysis. A two lane K type through truss bridge of 60 m span are modelled using finite element software (SAP 2000v14). A Rise of 6m was considered for both bridges. All the connections of the bridge members are riveted connections. Both the bridges were analyzed for IRC 70R and IRC AA wheeled loading. Deck is of concrete which are connected to the cross beams by shear connector. The cross beams are supported by main girders (longitudinal stringers). The material properties used for steel members and deck are of Fe 250 steel and concrete of M25. Steel truss elements, cross girders, diagonals, stringers, are modelled with frame elements. The frame element normally activates all the six degree of freedom at the both of its connected joints.

3.2 Finite Element Modelling of Steel Truss Bridge:

The Finite Element modelling of K type through steel truss bridge with a height of 6m and having span 60m modelled using SAP2000 version 14. The materials used are M25 grade concrete and Fe250 steel. The bridge roadway deck has been modelled with diaphragm constraints. Steel bridge elements top and bottom chords, cross girders, diagonals and stringers are modelled with frame elements. The frame element normally activates all the six degree of freedom at the both of its connected joints. By using the frame restraints we have developed simply supported condition to the bridge. By using the automatic frame mesh we ensure that proper connections were made at required nodes and cross girders.

Bridge member	Cross Section	Material property	Sectional area((mm^2)
Bottom Chord	ISMB 600	Fe 250	1560
Verticals and diagonals	2 ISMC400 with 2 plates of 400x8	Fe 250	2540
Stringers	ISMB 450	Fe 250	923
Top Chord	2 ISMC with 2 plates of 400x8	Fe 250	1110
Cross Girders	ISMB 400	Fe 250	785
Top Bracings	2 ISMC 250 with 2 plates of 300x6	Fe 250	1130

3.3 Analysis of Bridge:

Three basic analyses has been performed, static, modal and dynamic analysis using sap 2000 version 14.

3.3.1 Static analysis:

Static analysis of steel truss bridge has been performed using SAP2000 version 14. The loading considered in the study are of three types: IRC 70 R and IRC Class AA wheeled vehicle. The static analysis has been done to obtain the effect of dead load by assigning material properties of each component.

3.3.2 Modal analysis:

Modal analysis has been performed to determine the vibration modes of a structure and its natural frequencies. These modes are useful to understand the dynamic behaviour of the structure under dynamic loading. The modal analysis has been carried out using eigenvector analysis which has the basic equation as follow:

$$[\boldsymbol{K} - \Omega^{\mathbf{2}}\boldsymbol{M}]\phi = \mathbf{0}$$

In which K is the stiffness matrix, M is the diagonal mass matrix, Ω^2 is the diagonal matrix of eigenvalues, Φ is the matrix of corresponding eigenvectors (mode shapes).

3.3.3 Dynamic analysis:

The loading. In order to obtain accurate and reliable stress time histories, structural characteristics must be well most important factor to cause structural fatigue damage is stress fluctuation, which mainly induced by traffic identified. The following procedures are used for the present dynamic analysis of bridge.

In dynamic analysis of bridge, three different loadings, IRC 70 R, IRC Class AA wheeled vehicle were employed. The linear time-history due to a constant different vehicle speed in the range of 40 Kmph to 80 Kmph were obtained.

4.1 Fatigue due to Time Varying Loads

Fatigue is the process of accumulation of damage due to application of time varying stress. It can be expected to occur whenever a structure is subjected to time varying loads and in many situations may govern the design. Each time a load cycle is applied, an incremental amount of damage occurs. This damage is cumulative in nature and accumulation continues till the failure occurs. Our concern will be with the fatigue due to loads that vary in erratic manner and will be modelled by stochastic theory.

Both the material properties and the dynamic load process are important for fatigue evaluation, and should in most realistic cases are modelled as random phenomena. In order to relate a load sequence to the damage it inflicts to the material, the so called "Rainflow cycle" counting method is used, together with a damage accumulation model. The damage can then be related to the fatigue life. In the present work, Rainflow cycle counting method has been applied to identify the stress cycles responsible for damage accumulation in the structure. Standard algorithm has been used to calculate the expected Rainflow cycles, which are used for fatigue life evaluation.

4.2 Rainflow counting method

This method was reported in Japan in 1968 for the first time. This method obtained its name from an idea that water is flowing along a pagoda-shaped roof. The evaluation of random processes with reference to the fatigue of structures is based on the stress-time history shown in Fig 4.4

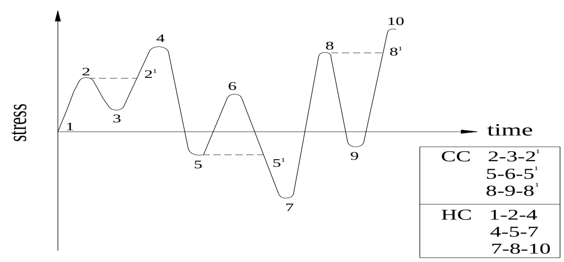


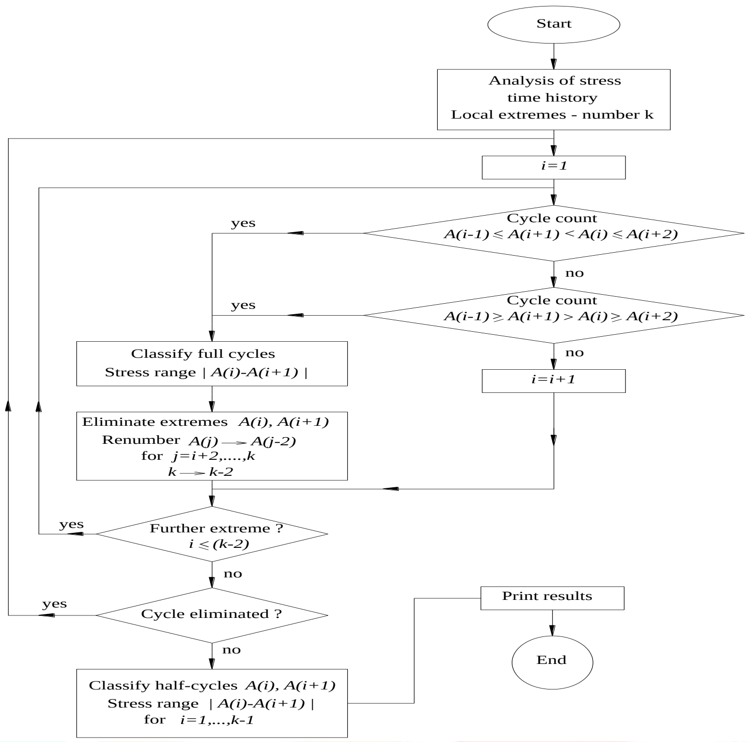
Figure IV-1 : The Figure shows Stress-time history (CC – Complete Cycles, HC –Half Cycles)

The fundamental assumption of the rainflow counting method is that fatigue damage due to small induced stress cycles may be added to the fatigue damage due to large stress cycles. If the cycle 1-4 in Fig. 4.4 is interrupted by a small complete cycle 2-3-2' the coordinate of point 2' is very near to the point 2 in Fig 4.4 and the material acts as if no interruption by an inserted cycle has taken place. Moreover, one complete cycle 2-3-2' has remained at disposal.

The rain-flow counting method evaluates the stress-time history in the same way as the material reacts the random process. It counts both the large amplitudes (half-cycle) and, separately, small inserted stress cycle (complete cycles). The "Rainflow Counting" method faithfully reflects the behavior of the material and characterizes its hysteresis. For this reason it is recommended for the evaluation of random stress-time histories with reference to fatigue of materials. Every time interval of stress is counted only once and the same result is obtained, if the evaluation proceeded in the opposite direction.

Systematic risk is the only independent variable for the CAPM and inflation, interest rate, oil prices and exchange rate are the independent variables for APT model.

The flow diagram of the computer program for the "Rainflow Counting" method is illustrated in Figure 4.6.



4.7 Procedure for Estimating Fatigue Life and Inspection Interval:

Fracture mechanics distinguishes three phases of the fatigue process which follow one another or even overlap:

- The phase of changes of mechanical properties.
- The phase of nucleation of fatigue cracks.
- The phase of crack propagation for which the stress conditions at the head of the crack are decisive. This phase terminates with fatigue fracture.

For civil engineering structures operating under normal traffic and temperature conditions the third phase lasts relatively for a long time. Generally a bridge with fatigue crack can serve for a long time in normal conditions. Therefore, the phase of crack propagation is given great attention.

The procedure for estimating fatigue life and inspection interval using fracture mechanics is as follows:

Paris- Erdogan derived a differential equation for the velocity of fatigue crack propagation in the second region of Fig 4.7

$$da/dn = C_0 (\Delta K)^m \qquad (4.10)$$

Where a – Crack length,
n – Number of stress cycles,
C_0, m – Material constants,
ΔK – Range of stress intensity factor which characterizes the stress conditions at the head of the crack.

C and m are functions of the material properties and microstructure, fatigue frequency, mean stress or load ratio, environment, loading mode, stress state and testing temperature.

The quantity ΔK depends on the direction of the applied force with reference to the direction of crack opening and is found from

$$\Delta K = \Delta\sigma_{ev} (\pi a)^{1/2} f(a) \qquad (4.11)$$

$\Delta\sigma_{ev}$ is equivalent stress range and can be calculated from

$$\Delta\sigma_{ev} = \left(\frac{1}{n} \sum_i n_i \Delta\sigma_i^k \right)^{\frac{1}{k}} \qquad (4.12)$$

$n = \sum_i n_i$ Is the number of all stress cycles

f (a) is crack growth function which is calculated for asymmetrical crack in a plate for flexural stresses depicted in figure 4.9. The crack growth function has been selected for plate under flexural stresses with asymmetrical crack given by Fryba [27].

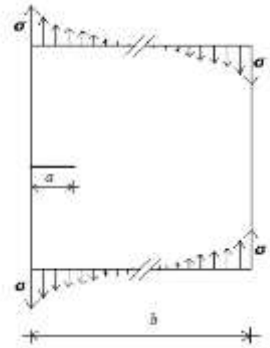


Figure IV-2 : The Figure represents Asymmetrical crack in a plate at flexural stresses, valid for a/b < 0.7

f (a) = 1.12-1.39(a/b) +7.3(a/b) ^{2}-13(a/b) ^{3}+14(a/b) ^{4}.

By substituting the above equations and rearranging the terms and introducing initial crack length a_0, equation (4.10) can be written as

$$dN = \frac{da}{C_0\left[\Delta\sigma_{ev}(\pi a)^{\frac{1}{2}} f(a)\right]^m \left(\frac{a}{a_0}\right)^{\frac{m}{2}}} \tag{4.13}$$

$$N - N_0 = \int_{a_0}^{a} \frac{da}{C_0\left[\Delta\sigma_{ev}(\pi a)^{\frac{1}{2}} f(a)\right]^m \left(\frac{a}{a_0}\right)^{\frac{m}{2}}} \tag{4.14}$$

Where N_0 corresponds to a_0

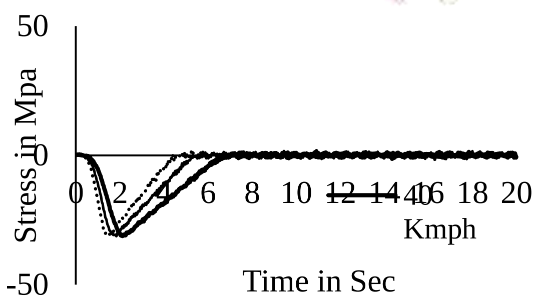
The above integration represents the number of cycles 'N' which increases crack length from a_0 to a.

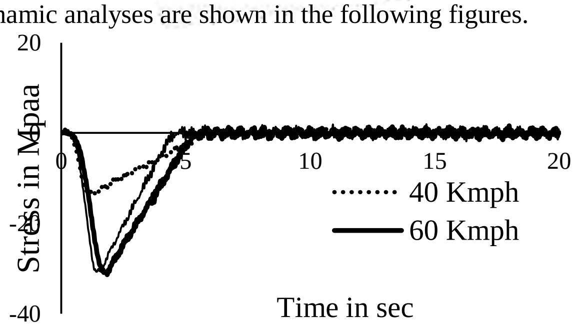
The result for the above integration has been taken from the reference [27]

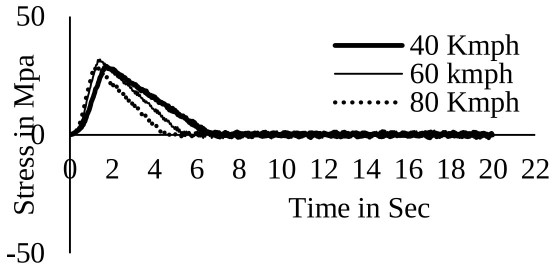
5.1 RESULTS AND DISCUSSIONS

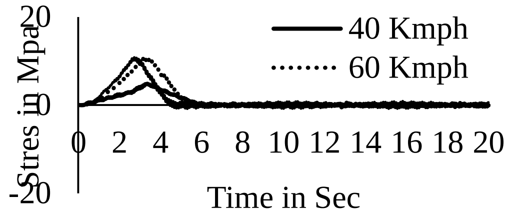
Dynamic Force Time Histories

Flexural stress time history at the mid span of the bridge has been considered for the evaluation of fatigue damage. The time-history is obtained by direct integration. The speed of the vehicle has been considered in the range of 40 km/h to 80 km/h. The vehicle loading employed here for determination of stresses are IRC 70 R wheeled loading and IRC AA wheeled loading. Force time history for different frame elements of *both* type of steel truss bridges obtained from the dynamic analyses are shown in the following figures.







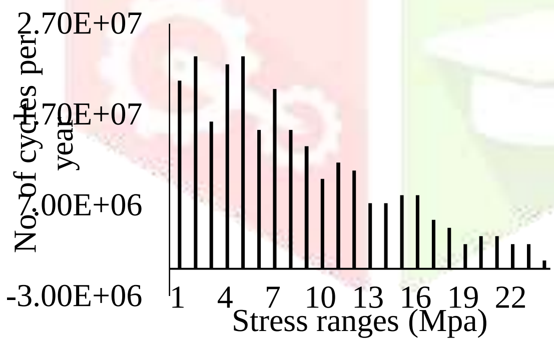


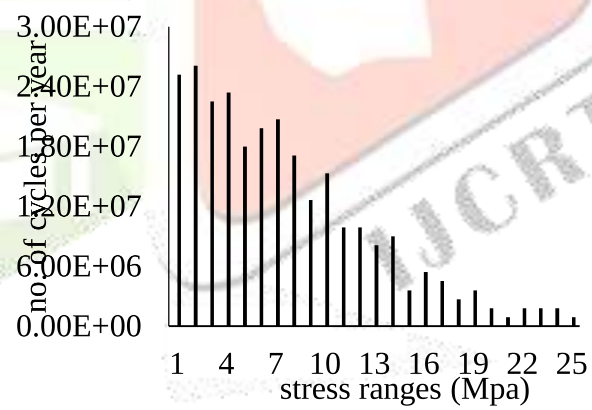
5.3 RESULTANT STRESSES AT CRITICAL CONNECTION:

We have observed from stress time history that the stringer-cross girder connection is very critical for the fatigue failure. From the observations and analysis the truss frame joints are highly protective against Fatigue failure as the magnitude of stress ranges are vary less at the joints. The stringer-cross girder connection is very critical for fatigue failure than the other connections of the frame elements in steel truss bridge [2]. At the stringer- cross girder connection, the flexural stresses are predominant in nature. Large amount of stress reversals are occurring at the stringer- cross girder joint. Stress reversal is the primary cause for the development of fatigue in the member, as it develops cyclic loading in the elements. In the Warren type two lane steel truss bridge, the stringer- cross girder connection is observed as the critical connection for fatigue and in K type truss bridge the stringer- cross girder joint is observed as critical connection for fatigue.

5.4 Stress Range Histogram:

The stress range histogram has been obtained for each loading by assuming uniform traffic for all the days in a year, number of cycles obtained from Rainflow analysis is simply converted in to annual cycles by using a constant multiplier (Average daily traffic x 365). The average daily traffic has been assumed as 2500 vehicles.
The stress range histograms at critical stringer-cross girder joint for fatigue in the both types of bridges are considered for the IRC 70R and IRC AA W loading, for the velocities ranging from 40 km/h to 80 km/h have plotted from figure 5.23 to 5.34.





5.1 Fatigue Life

5.5.1 Miner's Damage criteria using S-N curve approach

The Fatigue life at stringer-cross girder joints of K-type truss Bridges under IRC 70R and IRC AA wheeled loading for different velocities are tabulated in ble 5.1 to table 5.2. The S-N curve fatigue life parameter $K=2.34x10^{15}$ is taken from [4]. However, for three different values of m, fatigue life is calculated and presented in the tables. The values show that fatigue life decreases with increase in velocity for IRC 70R and IRC Class AA. However, the parameter m is very sensitive for calculation of fatigue life. For K type truss, the fatigue life for different velocities is found to be considerably high. This happens for the range of vehicle-speed considered in the study.

Velocity (Kmph)	Fatigue life in years (m=4)	Fatigue life in years (m=4.1)	Fatigue life in years (m=4.2)
40	536.8533	408.0738	309.9411
60	439.0537	329.5623	247.1431
80	365.8448	274.5819	205.9357

Table IV-1: Fatigue life in Stringer-cross girder joint of K type truss bridge joint under IRC 70R wheeled loading

Velocity (Kmph)	Fatigue life in years (m=4)	Fatigue life in years (m=4.1)	Fatigue life in years (m=4.2)
40	979.1188	753.9618	580.151
60	821.2146	630.0137	482.983
80	445.3753	334.7928	251.4771

Table IV-2 Fatigue life in Stringer-cross girder joint K type truss bridge joint under IRC AA wheeled loading

Velocity(Kmph)	Fatigue life in years	
	K truss	
	IRC 70R	IRC AA
40	Very high fatigue life and joint is over safe against fatigue Failure	Very high fatigue life and joint is over safe against fatigue Failure
60		
80		

Table IV-3 Fatigue life in truss frame joint in both bridges

6.1 Conclusions

In the present work, 3D model of common steel truss bridge (K type) have been analyzed for IRC loading-70R and AA wheeled. Natural frequencies, mode shapes and linear time history analysis were carried out using SAP 2000 commercial software. The dynamic stresses are then used to find out stress range histogram using cycle counting method. The annual traffic has been assumed in the analysis instead of practical data. Fatigue life calculations were carried out using Miner hypothesis.

(1) Natural frequencies and mode shape configuration are calculated.

(2) First mode is governed by lateral motion in both trusses and in subsequent mode independent/combination of bending and twisting are observed.

(3) Dynamic stresses in members are influenced by loading type and its speed. The higher magnitude of stress is found when speed of the vehicle is increased.

vibration.

6.2 Scope for future work

In this study fatigue analysis of steel bridges are carried out. Fatigue analysis can be carried out on concrete and pre stressed concrete bridges. The variation between the fatigue life derived from the S-N curve approach and the linear elastic fracture mechanics approach is considerably high. A reliability factor can be developed to correlate these two approaches. Non-linear fracture mechanics approach can be used for accurate damage analysis in the bridge members.

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