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Finite element analysis based fatigue life evaluation approach for railway bridges: a study in Indian scenario

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Abstract. Fatigue is a principal failure mode for steel structures, and it is still less understood than any other modes of failure. Fatigue life estimation of metal bridges is a major issue for making cost effective decisions on the rehabilitation or replacement of existing infrastructure. The fatigue design procedures given by the standard codes are either empirical or based on nominal stress approach. Since the fatigue life estimation through field measurements is difficult and costly, more researches are needed to develop promising techniques in the fatigue analysis of bridges through Finite Element Analysis (FEA). This paper aims to develop a methodology for the Fatigue life estimation of railway steel bridge using FEA. The guidelines of IIW-1823-07 were used in the development of the methodology. The Finite Element (FE) package ANSYS and the programming software MATLAB were used to implement this methodology on an Indian Railway Standard (IRS) welded plate girder bridge. The results obtained were compared with results from published literature and found satisfactory.

Keywords: fatigue analysis; finite element modelling; rain flow counting; structural hotspot stress

1. Introduction

A large number of old bridges in service are reaching their design service life. But it is impossible to replace all these bridges at once. Fatigue is a principal failure mode for steel structures, and it is still less understood than any other modes of failure. “In a study under the sponsorship of The American Society of Civil Engineers (ASCE), it was indicated that 80 - 90% of the failures in steel structures are related to fatigue and fracture” (Zhao *et al.* 1994). Fatigue life estimation of metal bridges is a major issue for making cost-effective decisions on the rehabilitation/replacement of existing infrastructure. Because of the increasing service loads and speeds, the fatigue life assessment is getting more relevance. This indicates the need for estimating the duration of these structures in useful service (Pipinato *et al.* 2012).

Different approaches exist for the fatigue analysis of welded joints. The method that demands the least work effort is the nominal stress concept followed by, Structural hot spot stress concept, Linear Elastic Fracture Mechanics (LEFM) concept and finally the most time consuming effective notch stress concept” (Pettersson and Barsoum 2012). Aygül *et al.* (2013) conducted a comparative

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study of different fatigue failure assessment methods of welded bridge details. The accuracy and applicability of Nominal stress method, Structural hotspot method and Notch stress method in fatigue analysis was investigated using different welded joints frequently used in steel bridges. The results indicated that the nominal stress method gave satisfactory results, despite of its simplicity. If the complexity in creating Finite Element (FE) models for numerical analysis is considered, the choice of the nominal method for fatigue analysis is fairly acceptable. It is difficult to distinguish the nominal stress in the vicinity of the welded joint through FEA. Till now, none of the available standards or guidelines have given clear instructions on how to determine the nominal stress from FE results (Hobbacher 2009). Hence the most suitable method to implement through FEA is the Hot spot stress approach.

The design approach in Euro Code is based on S-N curve. It can be seen that the provisions of Eurocode are based on a comprehensive study which takes into account the design life, type of connection, type of weld, location of joint, type of loading, traffic density, dynamic effects, no. of lanes, differential loaded lengths etc. Tables have been derived to easily work out the effect of these different factors. However, these tables are specific to the standard train loadings being used in Euro Nations. Within the framework of the Eurocodes, the member nations can determine the values of critical parameters. The member nations are allowed to use the codes with variations as per the practical conditions prevailing in their country (Goel 2005). Even though the major focus of the Eurocode is on the nominal stress method, guidelines on hot spot stress method are also incorporated.

Goel and Kumar (2006) presented a study of existing provisions of Indian Railway Standard Steel Bridge Code and the BS-5400, Part-10 in respect of fatigue design of Railway Bridges. Provisions of BS-5400 Part-10 which are based on S-N curve approach covers different loadings, loading situations, route Gross million Tones (GMT), class of connection etc. Standard type RU loading adopted in BS-5400 is found slightly heavier in comparison of IRS Modified broad Gauge (MBG) Loading, therefore the fatigue life assessment made based on RU loading on Indian conditions are expected to be on conservative side.

To allow for the effect of fatigue, the allowable working stress is determined from the Appendix G of IRS Steel Bridge Code for wide range of constructional details. The allowable stress ‘ P ’ depends on the ratio of minimum stress ‘σ_{min}’ to maximum stress ‘σ_{max}’ number of repetitions of stress cycle ‘ N ’, the method of fabrication and the type of connection. The code classifies the constructional details into seven classes considering the type of steel, type of fabrication and connection. All the details are designed such that the stress induced under design loads are within the allowable limits (Goel and Kumar 2006). The allowable stresses are the principal stress at the point under consideration. The bridge members are generally designed for 10 million cycles of stresses produced under the design load. Goel and Kumar (2006) points out the limitations of IRS approach as follows:

- There is no rational basis for adopting counts of 10 million number of cycles to determine the allowable stress levels
- The cumulative phenomena of fatigue is not reflected in the IRS procedure
- Stress-ratio procedure does not consider the effect of all stress ranges in a member
- No S-N curves are used in the present procedure
- Standard train load is represented as an equivalent uniformly distributed load. Thus, actual variation of stresses in a member due to passage of train is not counted

The fatigue design provisions of the Indian Standard steel design code, IS 800:2007 are based on the nominal stress approach. No indications are available on fatigue analysis using other approaches or FE modelling.

The International Institute of Welding (IIW) gives recommendations on fatigue of welded components and structures in the latest revision of the comprehensive code IIW-1823-07 (2008). This code is referred as *IIW recommendations* in this paper. The code is prepared by the Joint Working Group XIII-XV of International Institute of Welding under the chairmanship of A. Hobbacher. The new update includes the structural hot-spot stress Concept allowing now for an economic and coarser meshing in finite element analysis of welded structures (Hobbacher 2009). The IIW Recommendations give design S-N curves and clear guidelines on the determination of hotspot stress compared to other design codes.

The fatigue design procedures given by the standard codes were either empirical or based on nominal stress approach. Fatigue life estimation through field measurements is difficult and costly as most of the railway bridges are located at remote places. It has been identified that only few investigations have been conducted on fatigue analysis of bridges in Indian conditions. The fatigue analysis of bridges using FEA is still in the developing phase. More researches are needed to develop promising techniques in the fatigue analysis of bridges through FEA. This study attempts to develop a methodology for the Fatigue life estimation of Railway Bridge using FEA under actual moving train loads through an example of an IRS plate girder bridge of span 12.2 m.

2. Methodology

This paper uses the deterministic approach of fatigue analysis using Stress life method. The fatigue life was estimated by Parm-Miner damage summation using the endurance values and number of stress cycles obtained from a standard S-N curve and rainflow counting respectively. The type of stress used in the analysis was the structural hot spot stress. Fig. 1 illustrates the flowchart of the methodology adopted.

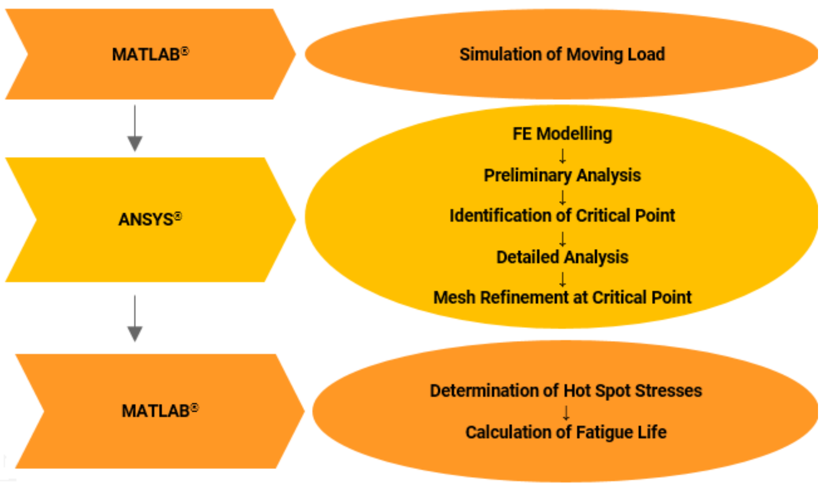


Fig. 1 Flowchart of the methodology of fatigue assessment

A simple MATLAB program was developed to simulate the moving train load. The program generates time histories of axle loads for any given speed input by the user. These time histories were exported to ANSYS to perform the dynamic analysis of the plate girder modelled in ANSYS. A preliminary moving load analysis was performed with most critical train loading to identify the critical location on the girder followed by detailed moving load analysis. The critical region was refined with finer meshing and the hotspot stress values were determined using a standard surface stress extrapolation technique through MATLAB coding for each train block loading. Another MATLAB code was used to count the hotspot stress cycles through rainflow cycle counting algorithm. This code also calculates the number of cycles for failure for different stress ranges using the standard design S-N curve given by IIW-1823-07 (2008) and accumulates the total damage through Parm-Miner rule.

3. Numerical modelling

The bridge considered in this study is the IRS welded plate girder bridge as per the Research, Design and Standards Organization (RDSO) drawing no. B16003. The span of the bridge is 40 feet (12.2 m) having a total weight of 10.26 t. The plate girder was modelled using ANSYS to perform the FE analysis. The bridge has two built up I girders placed parallel to each other and connected using lateral bracings. The web of the girders are stiffened with intermediate and end stiffeners. The intermediate stiffeners and bracings are Indian Standard Angles (ISA) while the web, flange and end stiffener are made up of Indian standard plate sections. The Material of the Girder is the Structural Steel conforming to the Indian standard IS 2062: 2011. Other properties are: Density 7850 kg/m^3, Yield strength 250 MPa, Young's modulus 200 GPa and Poisson's ratio 0.3. The cross sectional and material properties are shown in Tables 1 and 2.

Table 1 Cross sectional properties of the Girder

Component	Depth (mm)	Thickness (mm)
Top flange	550	25
Web	1235	12
Bottom flange	550	25
End stiffeners	1235	22
Intermediate stiffeners	Indian Standard Angles (ISA) 100 x 75 x 10	
Bracings	ISA 75 x 75 x 10	

Table 2 Material properties of the Girder

Property	Value
Material	Structural steel (IS 2062: 2011)
Density	7850 kg/m 3
Yield strength	250 MPa
Young's modulus	200 GPa
Poisson's ratio	0.3

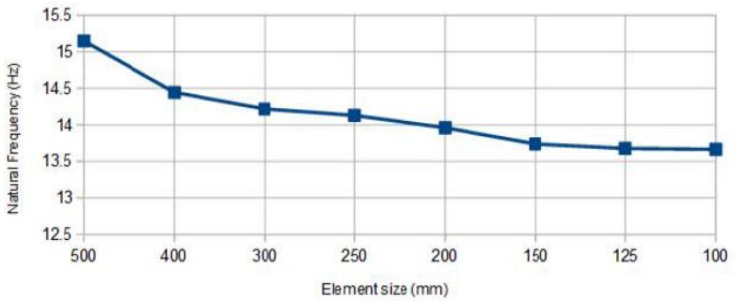


Fig. 2 Mesh convergence study based on modal analysis

3.1 Meshing and boundary conditions

The bridge model is composed of plates whose thickness is small when compared to other two dimensions. If one of the dimensions of the geometry to be meshed is very small compared to the other two, 2D elements are preferred for the FE modeling (Gokhale 2008). The bridge model was meshed using shell and beam elements. All plates were modeled using the shell elements and the intermediate stiffeners and bracings using beam elements.

A sensitivity study with different element sizes was carried out to find out the most computationally efficient mesh. The convergence of first natural frequency values from the modal analysis result was the basis of this study. Fig. 2 shows the mesh convergence study based on modal analysis. The frequency started converging at an element size of 150 mm along the length of the girder.

The bridge is supported on the piers/abutments with a bearing plate placed between them. The bearing plates are welded to the bottom flanges of the girders and rigidly bolted into the abutment concrete. The bearing plates were modelled using shell elements and the welded connection with the flange was represented by bonded type contact in ANSYS. The bottom face of the bearing plates were given fixed boundary condition. These boundary conditions were adopted after making close observations of the actual field boundary conditions of the Indian railway plate girder and discussions with the railway bridge engineer. The train load applied on the rail is transferred to the bridge through two channel steel sleepers placed back to back. Bearing plates are sandwiched between the sleepers and top flange of the bridge. To simulate this field condition, bearing plates were modeled on top of the top flange and the axle loads were applied as pressure values acting downwards on this plates. A standard sleeper density of M+7 was chosen which gives 20 sleepers on the bridge span (Saxena and Arora 2012).

3.2 Fatigue loading

The design loads for railway bridges are defined by IRS Bridge Rules – 2008 : Rules Specifying The Loads for Design of Super-structure and Sub-structure of Bridges and for Assessment of The Strength of Existing Bridges. The IRS MBG loading specifies the axle loads and spacings in respect of the loco only and therefore the effects of axle loads and spacings in respect of wagons are not specifically considered. For conducting detailed analysis of fatigue strength of bridge components, the effects of individual axles are required to be accounted for.

International standards like Eurocodes and BS have Load models available for fatigue assessments.

The fatigue load model developed by Goel (2007) for Indian railway bridges was used in the present study. This load model can be used for generating the route specific stress range & frequency histograms and the cumulative fatigue damage can be worked out using Palmgren-Miner rule. Goel (2007) classified the Indian railway traffic into four: heavy freight traffic, mixed traffic lines with heavy traffic, suburban traffic and mixed traffic lines with light traffic. Among these, the suburban traffic was chosen in this study considering that it has the least variety of trains. The most intensively used suburban line in India is on Western Railway, Bombay-Virar, which has an existing traffic of 200 suburban trains per day and 10 other passenger trains. The two passenger trains are composed of 2 locomotives each with 22 and 26 coaches respectively. The average speed of train in this line, 50 kmph was chosen as the speed for the present study. Table 3 gives the details of the load model.

The fatigue strength of the bridge will be governed by the suburban train consisting of Electric Multiple Units (EMU). Each unit have engine attached with it. Since the trains are composed of repeated units, the identical units can be grouped into a single loading block as shown in Table 4. The vertical arrows represents the axle loads in tons and the axle spacings are given in mm. Since we assume linear damage accumulation, the total fatigue damage caused by these units will be the sum of individual blocks. Table 5 gives the total number of repetitions of each block loading to be used in the damage accumulation.

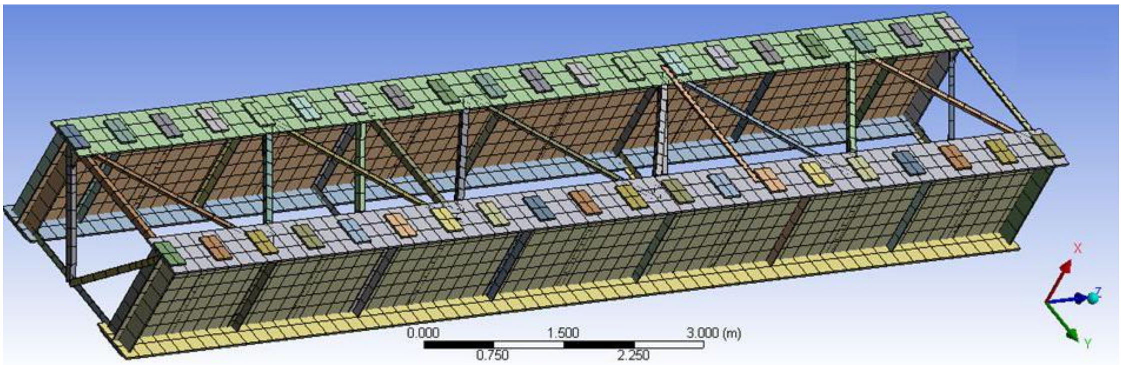


Fig. 3 Meshed model

Table 3 Fatigue load model for suburban traffic (Goel, 2007)

Train	Train type no. (As per Goel 2007)	No. of trains per day
2 Loco_1 + 22 Coach_2	2	5
2 Loco_1 + 26 Coach_3	3	5
EMU	4	200
Total		210

Table 4 Axle load diagrams for different blocks of load

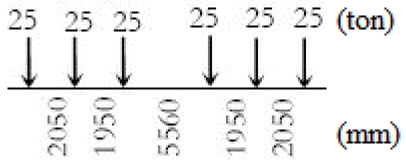
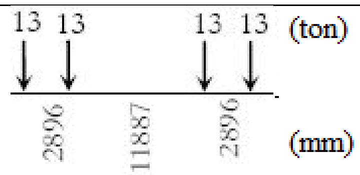
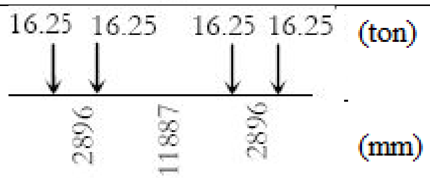
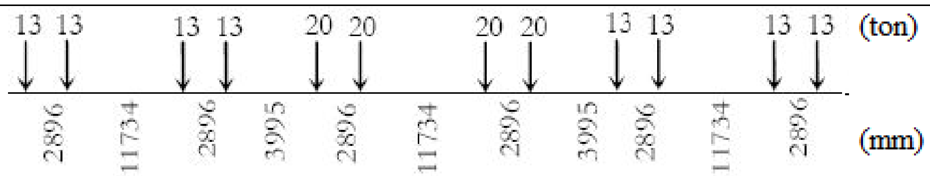
Block name	Load diagram
Loco_1	
Coach_2	
Coach_3	
EMU	

Table 5 Load repetitions per year

Block name	Projected no. of repetitions per year
Loco_1	7300
Coach_2	40150
Coach_3	47450
EMU	292000

4. Moving load analysis

According to Niemi *et al.* (2006), the determination of structural hot spot stresses using FE Analysis can be performed by creating a coarse model to identify the hot spot area followed by sub modeling this area, using the nodal displacements or nodal forces from the original model as loading at the boundaries of the sub-model. An alternative method is to refine the original element mesh in the hot spot region. In the present study the second approach was adopted.

Imam *et al.* (2007) used FE principal stress histories to apply miner's rule in fatigue damage calculation of steel railway bridge. Since the maximum principal stress, σ_1 gives the maximum positive tensile stress components of the three dimensional stress state, it can give more realistic fatigue damage estimates. IIW recommendations and Eurocode 3 advise to use the maximum principal stress as the appropriate stress component (Heshmati 2012).

The EMU trains are assumed to be contributing the majority of the fatigue damage to the bridge under the suburban traffic loading mentioned in the previous section. Hence, a preliminary analysis was carried out under this full train EMU loading on the bridge model to find out the most critical location on the bridge. The contour plot of maximum principal stress for the web plate is given in the Fig. 4. The results revealed that the critical location is the web to flange butt weld connection near the support.

Fig. 5 shows the time history of maximum principal stress (absolute values for the whole bridge) from the preliminary analysis carried out under full train loading on the bridge model. This time history was found to be following a repeated fashion. This is due to the fact that the train load is the combination of identical repeating units of locos and coaches. So, instead of loading the bridge with full train, individual block loading is enough to capture the dynamic response of the bridge in the detailed analysis.

The vector plot of maximum principal stress at the critical location shows that its direction is predominantly perpendicular to the assumed weld toe at the support region (Fig. 6). Hence, the use of maximum stress value for the extraction of hotspot stress is justified. At the next stage the model was refined according to the IIW recommendations in order to extract the structural hot spot stress values under detailed moving load analysis.

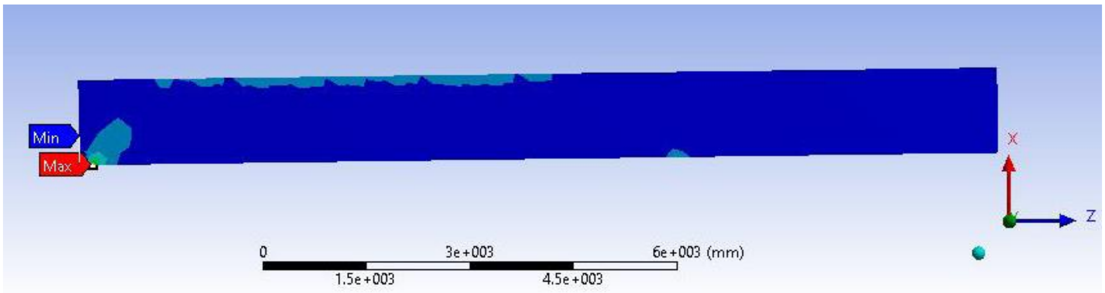


Fig. 4 Result showing the critical location on the web plate

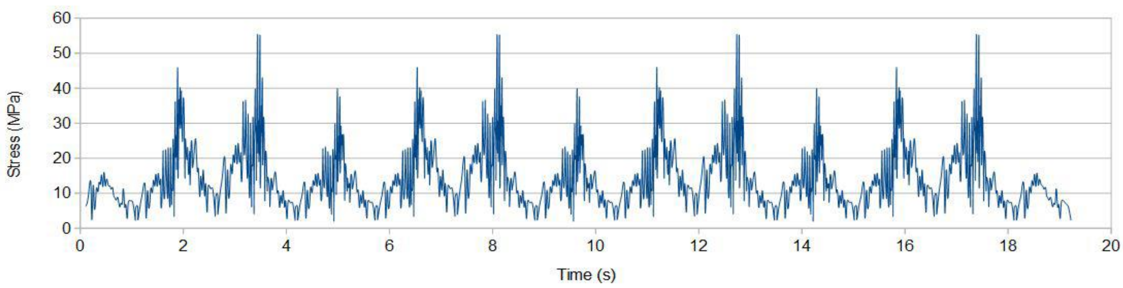


Fig. 5 Time history of maximum principal stress under full EMU loading

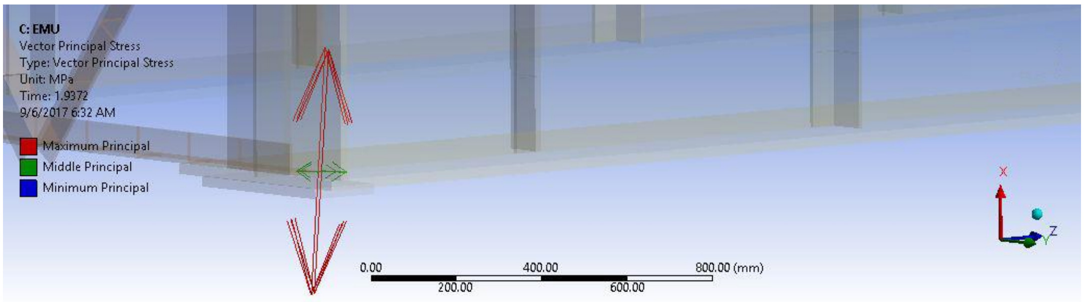


Fig. 6 Vector plot of maximum principal stress at the critical node

5. Determination of hot spot stress

The SHSS comprises the stress rises due to all geometric parameters except the notch effect caused by the weld profile. The FEA results are found to be extremely sensitive to the mesh size and type of finite elements at the weld toe region due to the abrupt stress concentration. Therefore, a comprehensive guideline for the basis of the structural hot spot stress evaluation using finite element method is required. IIW-1823-07 (Recommendations for fatigue design of welded joints and components) allows to determine the hot spot stress by extrapolating the surface stress at certain reference points to the weld toe.

The IIW recommendation defines two types of hotspots according to their location and orientation- *Type a* in which the weld is located on plate surface and *Type b* in which the weld is located at plate edge. The procedure is first to identify the type of hotspot. Then establish the reference points and determine the structural hot spot stress by extrapolation to the weld toe from the stresses of those reference points using the extrapolation equations given by the code. The hotspot in this problem is identified as *Type a* . Welded joints of plates can be modelled with solid or shell elements with or without modelling the weld. IIW recommendations provide linear extrapolation equations for both cases. When using shell elements for evaluation of welded structures, the welds are usually not modeled. The approach used in this study is to model the joint using shell elements without a weld.

5.1 Surface stress extrapolation

IIW recommendations allow fine or coarse meshing densities for shell and solid elements. It is a confirmed fact that the FE models with fine meshing generally yield more accurate results. A relatively coarse mesh can also be used to determine the structural hot spot stress. A FE model with a relatively coarse mesh is advantageous over the same model with a fine mesh as it requires less computational power. When using a coarse mesh for the FE model, the recommended elements sizes should be strictly followed and no alterations are permitted. These recommendations are summarized in Table 6.

In the present study the type of hot spot was '*Type a'* and the element used is shell. From Table 6, the size of shell elements to be used in relative coarse meshing is equal to the thickness of plate. To satisfy the IIW guidelines, the plate was refined near the hotspot area as shown in Fig. 8(a). Since the thickness of the web plate was 12 m, the element size at the refined area was chosen the same. Surface stress extrapolation of '*Type a*' hotspot is illustrated in the Fig. 8(b). Eq. (1) gives the expression for surface stress extrapolation by IIW-1823-07.

$$\sigma_{hs} = 1.5\sigma_{0.5t} - 0.5\sigma_{1.5t} \tag{1}$$

Where, σ_{hs} : Hot spot stress, $\sigma_{0.5t}$: Stress at a distance of 0.5t from the weld toe, $\sigma_{1.5t}$: Stress at a distance of 1.5t from the weld toe and t : element size = 12 mm.

In order to use the Eq. (1) for stress extrapolation, the stress values at the midpoint of the element is required. ANSYS does not allow the user to directly extract the stress time history at mid node points. It gives only the time history at nodes. Hence it becomes essential to interpolate the stress values at the extraction points *0.5 t* and *1.5 t* from the nodal values. The shape function of the 4 noded quad shell element shell181 in ANSYS is given by Eq. (2)

$$u = \frac{1}{4}\left(u_i(1-s)(1-t) + u_j(1+s)(1-t) + u_k(1+s)(1+t) + u_l(1-s)(1+t)\right) \tag{2}$$

Table 6 IIW recommendations for meshing and extrapolation (IIW-1823-07)

Type of model		Relatively coarse mesh		Relatively fine mesh	
		Type a	*Type b*	*Type a*	*Type b*
Element size	Shells	t x t	10 x 10mm	≤ 0.4 t x t	≤ 4 x 4 mm
	Solids	t x t	10 x 10mm	≤ 0.4 t x t	≤ 4 x 4 mm
Extrapolation points	Shells	0.5 t & 1.5 t (mid-side points)	5 & 15 mm (mid-side points)	0.4 t & 1.0 t nodal points	4, 8 & 12 mm nodal points
	Solids	0.5 & 1.5t (surface center)	5 & 15 mm (surface center)	0.4 t & 1.0 t (nodal points)	4, 8 & 12 mm (nodal points)

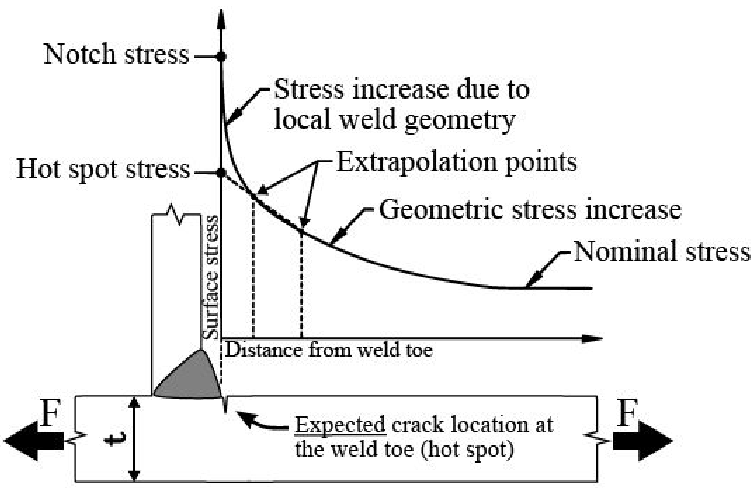
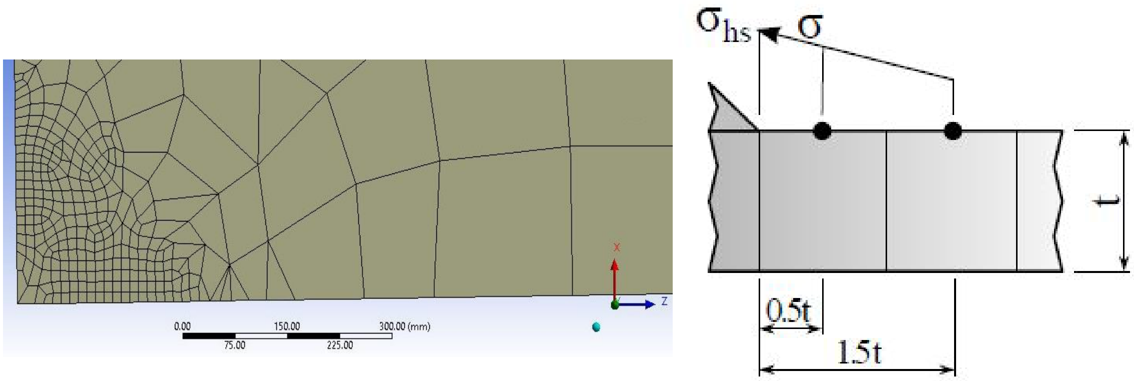


Fig. 7 Stress Distribution near the Weld Toe (Heshmati 2012)



(a) (b)

Fig. 8 (a) Refined meshing of the web plate near the hotspot area and (b) Surface stress extrapolation of *Type a* hotspot (Heshmati 2012)

Where u is the value at any point inside the element, u_i , u_j , u_k and u_l are the values at nodes i , j , k and l respectively and s, t are the natural coordinates. The interpolation point lies at the midpoint between the nodes i and l. The natural coordinates of this point are s = -1 and t = 0 respectively. The stress values has to be interpolated for all time points in the time history. The time history of SHSS was created using the surface stress extrapolation technique of IIW-1823-07 from the stress histories obtained from ANSYS results for each train load cases with the help of MATLAB coding. This time histories were then used to calculate the fatigue damage using a suitable S-N curve.

6. Calculation of fatigue life

The determination of stress ranges ($\Delta\sigma$) and corresponding no. of cycles (n) from the SHSS time histories was carried out using rainflow cycle counting algorithm. The ASTM standard E 1049-85 : Standard practices for cycle counting in fatigue analysis (1997), provides the guidelines for cycle counting in fatigue analysis under random loading conditions. *MATLAB central* provides a toolbox, RAINFLOW which includes rainflow cycle counting algorithm prepared for using in the MATLAB environment. The algorithm code has been written by Nieslony (2003) according to the standard ASTM E 1049–85 (1997). This MATLAB code was modified to suit the present problem to calculate the fatigue damage according to IIW recommendations.

The choice of appropriate SHSS design S-N curve is one of the main issues regarding the fatigue life estimation of plate-type welded joints. According to the conventional assumption, the S-N curves terminate at a fatigue limit, below which failure will not occur, or in which case the curve becomes horizontal. Traditionally, this Constant Amplitude Fatigue Limit (CAFL), also referred as ‘knee point’, is defined in terms of the corresponding number of cycles on the S-N curve, $N=10^7$ being the most common assumption. The slope of the curve is 3 below this point. However, new experiments indicate that a constant amplitude fatigue limit does not exist and the S-N curve should continue on the basis of a further decline in stress range at a slope of m= 22. Fig.

9 gives the Modified resistance S-N curves of steel defined by IIW-1823-07 for Palmgren - Miner summation.

When the fatigue life estimation is carried out based on the SHSS approach, fatigue design classes (FAT) are attributed to details with reference to the weld type. IIW recommendations defines nine weld types for fatigue life estimation using SHSS approach. The web to flange connection in the plate girder bridge is a double bevel butt weld. The most suitable fatigue design class for this type of joint as per IIW recommendations is FAT 100. For this FAT class the knee point value is given as 58.5 MPa. The fatigue resistance of a welded joint is limited by the fatigue resistance of the base material. The FAT class of the base material here is FAT 160 and is expected to give higher fatigue resistance than the joint. The general equation of an S-N curve is given by,

$$N = C.\Delta\sigma^{-m} \tag{3}$$

Where $\Delta\sigma$ = Stress range and m is the slope of the curve. C is a constant found experimentally which accounts for different factors affecting the fatigue. The S-N curve used for this problem is the two slope curve proposed by IIW recommendations. The Table 7 gives the design values for the S-N curve for FAT 100.

The expressions for the IIW S-N curve for FAT 100 are given in Eqs. (4) and (5). The N values from the S-N curve and the no of cycles n from the cycle counting results were used in Eq. (6) for linear damage accumulation according to Parlmgren-miner rule.

$$N_{\mathrm{i}} = 2.000 \,\mathrm{x}\, 10^{12}\ \Delta\sigma^{-3}\ , \mathrm{for}\ \Delta\sigma \geq 58.5\,\mathrm{MPa} \tag{4}$$

$$N_{\mathrm{i}} = 6.851 \,\mathrm{x}\, 10^{15}\ \Delta\sigma^{-5}\ , \mathrm{for}\ \Delta\sigma \leq 58.5\,\mathrm{MPa} \tag{5}$$

$$D = \sum_{\mathrm{i}=1}^{j} \frac{n_i}{N_i} \tag{6}$$

The fatigue damage estimation was done for the SHSS time histories for all train loads. The damage caused by each train loads was added to get the total damage due to all trains. The fatigue life of the plate girder bridge was then calculated by taking the inverse of the total damage. MATLAB coding was used to implement this procedure. The fatigue life of the plate girder bridge were estimated as 41 years 1 month and 25 days.

Rao *et al.* (2013) conducted a fatigue life analysis of welded steel plate railway bridge girders designed as per Indian railway standards using S-N curve approach in frequency domain. The fatigue life estimate of 12.2 m span plate girder bridge by Rao *et al.* (2013) is found to be 45 years. “It is reported that the MBG loading specified by IRS is comparable to RU loading specified in BS 5400: Part 10” (Rao *et al.* 2013). Hence Rao *et al.* (2013) had used the RU loading in their study. The fatigue load model used in the present study is based on IRS MBG loading. The geometric properties of the bridge analysed by Rao *et al.* (2013) are slightly different from the bridge analysed in the present study. Even though the loading conditions and bridge model are not exactly the same for both works, a reasonable comparison may be acceptable.

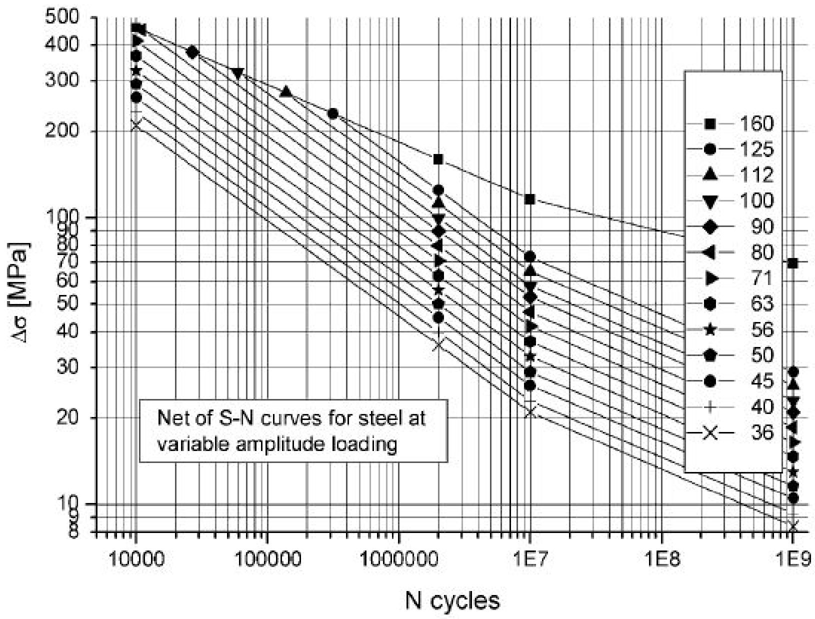


Fig. 9 Modified resistance S-N curves of steel for Palmgren - Miner summation (IIW-1823-07)

Table 7 Design values for the S-N curve

FAT class		100
Stress at 2×10^6		100 MPa
Stress at knee point (@ 10 7 cycles)		58.5 MPa
Constant, *C*	For stress ranges above knee point	2.000×10^{12}
	For stress ranges bellow knee point	6.851×10^{15}
Slope, m	For stress ranges above knee point	3
	For stress ranges bellow knee point	5

5. Conclusions

A methodology for the Fatigue life estimation of Railway Bridge using Finite Element Analysis (FEA) was developed in the present study. The guidelines of IIW-1823-07 were used in the development of the methodology. The FE package ANSYS and programming software MATLAB were used to implement this methodology on an IRS plate girder bridge of span 12.2 m.

The fatigue damage was calculated from the stress ranges obtained from the moving load analysis results of plate Girder Bridge under standard train loads. The preliminary analysis results revealed that the critical location is the web to flange butt weld connection near the support.

The fatigue life of the plate girder bridge was calculated using Parlmgren-Miner damage accumulation model through MATLAB coding. The estimated fatigue life was as 41 years, 1 month and 25 days. The results obtained were compared with results from published literature and found satisfactory. Considering the nature of the fatigue phenomenon and the unpredictability associated with it, the results presented in this paper based on the numerical study need to be validated through actual field observations.

Many factors like corrosion, over loading, variations in train speeds, braking load effects, variations in train traffic, variations in train composition etc. which affects the fatigue, are not considered in the present study.

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