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Multiaxial Fatigue Damage Assessment of Welded Connections in Railway Steel Bridge using Critical Plane Approach

Praveen K R[a]*, S S Mishra[a], Prasad Babu[b], Andrea Spagnoli[c], Andrea Carpinteri[c]

[a]*Department of Civil Engineering, NIT Patna 800005, India*
[b]*Deustche Bahn Engineering and Consulting Gmbh, Bengaluru 560001, India*
[c]*Department of Engineering and Architecture, University of Parma, Parma 43124, Italy*

Abstract

As the number of heavy railway traffic load increased, concern over the accurate and actual fatigue damage of the bridge is intensified. Especially for bridges which were designed for light traffic load. The fatigue damage assessment of steel bridge connections is usually based on notion of uniaxial S-N curves given in the codes of practice. Until now, there is no consensus on a method which can precisely consider non-proportional multiaxial loading. The objective of this paper is to examine the applicability and appropriateness of the critical plane approach-based C-S criterion to perform the fatigue damage assessment in welded connections in railway steel bridge. A regular U trough railway steel bridge is analyzed using finite element software ANSYS 17.2 for standard railway traffic. The averaged principal stress directions determined through appropriate weight functions are used to orient the critical plane. Prediction of fatigue damage is performed through an equivalent stress represented by a quadratic combination of the normal and the shear stress components acting on the critical plane. Applicability of the C-S criterion is studied by assessing the fatigue damage of critical welded connections and comparing with the λ - coefficient and cumulative damage method calculated according to Eurocode EN1993-1-9.

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Keywords: Multiaxial high cycle fatigue, Carpinteri-Spagnoli criterion, Railway bridge

* Corresponding author. Tel.: +91 8904474487
E-mail address: prawinelazar@gmail.com

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1. Introduction

Due to enhanced demand in Reliability, Availability, Maintainability and Serviceability (RAMS) of structural systems, there is an increase in demand to understand and estimate the precise fatigue damage in bridges. An advanced fatigue design methodology is required to recognize potential critical details introduced by specific design changes. So far, there is no precise and robust fatigue damage estimation tool particularly for welded details. Especially in bridges, stresses developed at welded connections are always multiaxial and non-proportional. Fatigue assessment based on the critical plane approach is generally accepted to be more accurate for multiaxial non-proportional loading.

Nomenclature

C_a	shear stress amplitude acting on the critical plane	δ	angle between the direction $\hat{1}$ and the normal w to the critical plane
m	slope of SN curve	$\phi(t)$ $\theta(t)$ $\psi(t)$	Euler angles at time instant t
L	Taylor's critical distance	γ_{Ff}	partial safety factor for applied stress range
N	number of cycles to failure	γ_{Mf}	partial safety factor for detail category
N_c	endurance limit for detail category $\Delta\sigma_C$	λ	damage equivalent factors
n_i	number of cycles of individual cycle block	λ_{max}	1.4
N_i	endurance limit of individual cycle block	ϕ_2	dynamic magnification factor as per Annex D, section D.1 in EN 1991-2:2003(E)
N_o	reference loading cycles, 2×10^6		
Na	normal stress amplitude acting on the critical plane	$\Delta\sigma_{71}$	stress range due to the load model LM71
		$\Delta\sigma_C$	detail category
W(t)	weight function at time instant t	$\Delta\sigma_{c\text{-}s}$	equivalent normal stress range
W	summation of weight functions W(t)	$\Delta\sigma_i$	calculated stress range
$\hat{1}$ $\hat{2}$ $\hat{3}$	averaged principal stress directions	ΔK_{th}	threshold stress intensity
$\hat{\phi}$ $\hat{\theta}$ $\hat{\psi}$	averaged Euler angles	$\sigma_1(t)$	principal stress at time instant t
z	number of reversals for reduced stress components	$\sigma_{af\text{-}1}, \tau_{af\text{-}1}$	fatigue limit under fully reversed normal stress and shear stress respectively

There are several multiaxial fatigue damage models in the literature which propose different damage parameters. In general, critical plane models require scanning over all planes intersecting the surface either orthogonally or at some inclination for maximum value of damage parameter. Also, the stress analysis has to be performed for each time step considered in the simulation of the train crossing. This complex and cumbrous task is simplified in the C-S criterion by applying some weight functions and linking the critical plane with mean principal stress directions.

2. Description of railway steel bridge

The superstructure of single track railway bridge consists of steel U trough, made up of two main longitudinal girders spanning 27.8m between abutments and inverted T cross girders connecting the main girders for the entire length of the deck. It is most suitable for medium span bridges, where there is shallow depth between the trafficked surface and the clearance level underneath. Therefore, steel U trough railway steel bridges are widely preferred rather than deck type plate girder bridges. The bridge is simply supported and resting on elastomeric bearings. The dynamic analysis has been carried out using finite element software ANSYS 17.2.

The height of the main girder and cross girder is 1.53 m and 0.4 m respectively (see Figure 1). Width of the bridge measures 3.82 m and thickness of deck plate is 20 mm. Cross girders are spaced at 0.67m centre to centre as is shown in the Figure 2. Transverse stiffeners are provided at a spacing of 1.33 m. Lateral torsional buckling of the bridge is restrained by U-frame action of steel trough. All structural members are made up of high-strength low-alloy European standard structural steel S355 with a yield strength of 355MPa and ultimate strength of 611MPa. All supports are rotation free and translations are allowed along the horizontal directions to accommodate longitudinal and transverse movements arising due to temperature, breaking-traction, wind, nosing, centrifugal actions, etc.

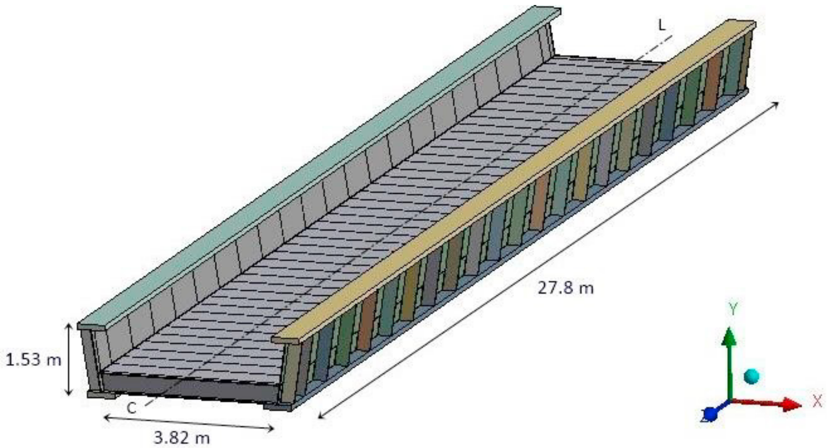


Figure 1. 3D view of global numerical model of the bridge

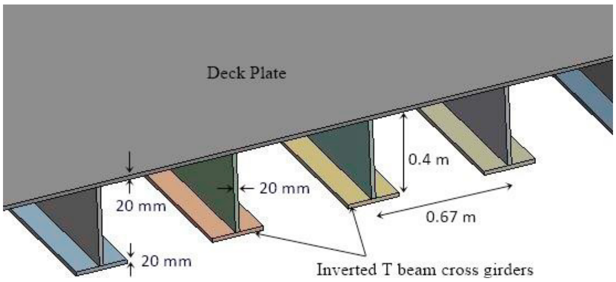


Figure 2. Longitudinal section of bridge showing inverted T cross girders

3. Fatigue load models

3.1. Fatigue load model for λ – coefficient method

Fatigue load model LM71 represents the static effect of railway traffic loads calibrated using the International Union of Railways (UIC) and European railway traffic data. LM71 load model is used for preliminary fatigue assessment. For continuous bridges, SW/0 and SW/2 are used for standard and heavy traffic, respectively. The axle load configurations for LM 71 are given in *EN 1991-2:2003* [5] clause 6.3.2 as shown in Figure 3.

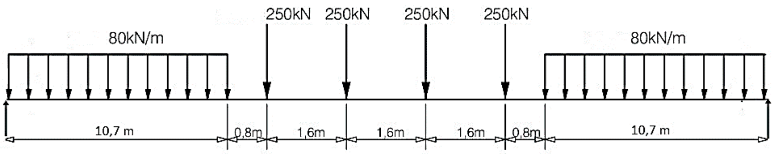


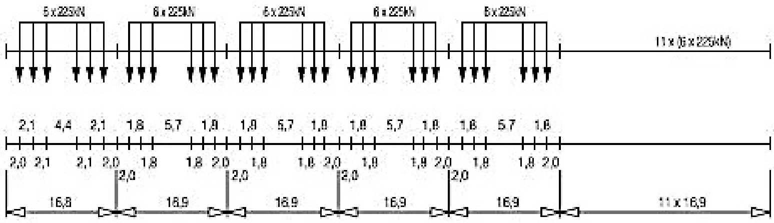
Figure 3. Fatigue load model LM71

3.2. Fatigue load model for cumulative damage method

According to *EN 1991-2:2003* [5], *Annex D* the fatigue load model is given by set of 12 trains (including passenger and freight trains) and are categorised as “standard”, “heavy” and “lightweight” traffic mixes. The fatigue damage of a detail depends on the traffic spectra that pass over the bridge during its service life and on its detail category. This depends on whether the railway line predominantly carries passenger cars or freight cars. If the traffic mix does not represent the real traffic, an alternate traffic spectra specified in national annexes should be used.

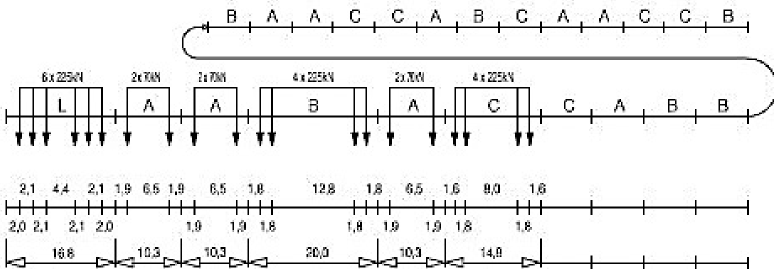
Type 5 Locomotive-hauled freight train

$\Sigma Q = 21600\text{kN}$ $V = 80\text{km/h}$ $L = 270{,}30\text{m}$ $q = 80{,}0\text{kN/m'}$



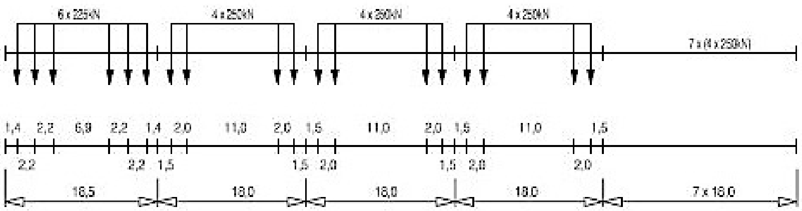
Type 6 Locomotive-hauled freight train

$\Sigma Q = 14310\text{kN}$ $V = 100\text{km/h}$ $L = 333{,}10\text{m}$ $q = 43{,}0\text{kN/m'}$



Type 11 Locomotive-hauled freight train

$\Sigma Q = 11350\text{kN}$ $V = 120\text{km/h}$ $L = 198{,}50\text{m}$ $q = 57{,}2\text{kN/m'}$



Type 12 Locomotive-hauled freight train

$\Sigma Q = 11350\text{kN}$ $V = 100\text{km/h}$ $L = 212{,}50\text{m}$ $q = 53{,}4\text{kN/m'}$

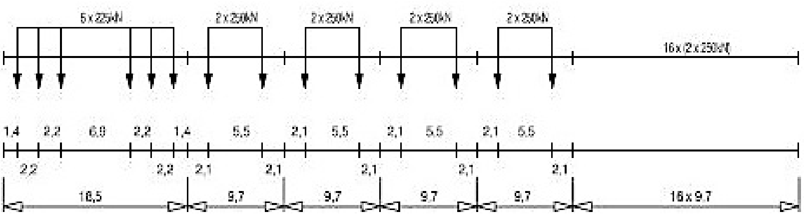


Figure 4. Axle load and axle configuration of train types 5, 6, 11 and 12 in heavy traffic mix

The fatigue analysis is performed based on the heavy traffic mix. Heavy traffic mix is composed of four different combinations of fatigue trains as is shown in Table 1. The axle load configuration is represented in Figure 4.

Table 1. Train composition of heavy traffic mix as per *EN 1991-2:2003 [5]*, *Annex D*

Train type	Number of trains/day	Mass of train (t)	Traffic volume [10^6/year]
5	6	2160	4.73
6	13	1431	6.79
11	16	1135	6.63
12	16	1135	6.63
Total	51		24.78

4. Numerical modelling of bridge

Linear elastic stress analysis of the railway bridge is performed by numerical finite element approach using ANSYS 17.2. Global finite element model of the bridge is generated using hexahedron 8-node solid elements (termed SOLID185 in ANSYS). There are 33,346 elements and 28,336 nodes. Materials are assumed to be linear elastic and isotropic with modulus of elasticity 200GPa and Poisson’s ratio 0.3. The contact between structural members is modelled using surface to surface bonded option. In particular, the CONTA174 and TARGE170 elements are used to model the contact and target surfaces forming a contact pair. With this global numerical model a prior identification of critical welded detail is performed using the λ – coefficient method and cumulative damage method.

4.1. Sub modeling

Sub modelling is an approach that allows us to solve a small part of a bigger model, with more refined meshes and results. In this paper, a refined local finite element model of critical welded detail is created by sub modelling approach using 3D solid finite elements. Multi Point Constrains are used to define the boundary conditions. In ANSYS 17.2, the procedure is to identify the faces where the cut is made from global numerical model. These faces are designated as cut boundary faces and the nodes on these faces are known as “cut boundary nodes”. Critical welded detail is then simulated by interpolating the displacement histories from the global numerical model against these cut boundary nodes. The furthermost benefit of this method is that there is no necessity of fine meshing the entire global model in order to cover the very fine details such as welds, bolts, hole etc.,

5. Fatigue damage assessment

It is the most common practice to distinguish the global methods and local methods of fatigue assessment of welded joints. In this paper, the global approaches such as λ coefficient method and cumulative damage method are used in conjunction with established S-N curves as a preliminary fatigue analysis. High computational cost is mitigated by performing a preliminary fatigue assessment in order to locate the potential critical weld. Subsequently, a detailed local approach in conjunction with finite element analysis is performed on the critical welded detail. The outline of approach is shown in Figure 5.

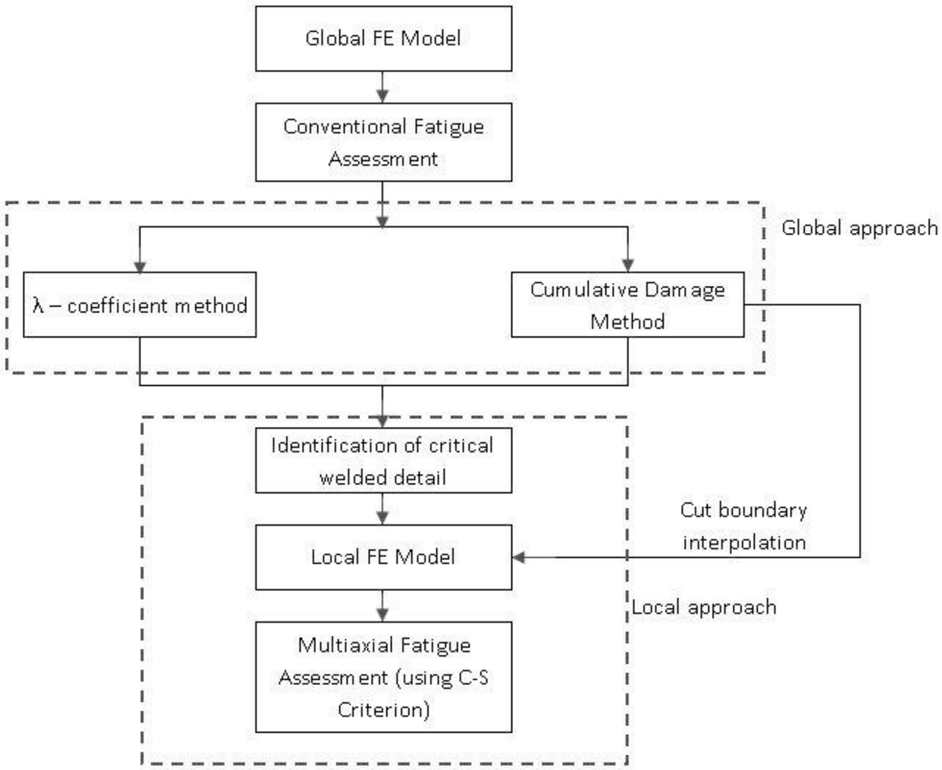


Figure 5. Flowchart of methodology

5.1. λ – coefficient method

The basic concept of this method is that the variable amplitude loading generated by railway traffic is simplified to equivalent constant amplitude loading using λ-factors. The nominal stresses obtained from ANSYS 17.2 using load model LM71 as per *EN 1991-2:2003* [5] are modified by four λ-factors and expressed as an equivalent stress range corresponding to 2 x 10^6 cycles, $\Delta\sigma_{E,2}$. Consequently, fatigue assessment is simplified as a comparison between the equivalent stress range at 2 x 10^6 cycles and the detail category.

$$\gamma_{Ff}\lambda\phi_2\Delta\sigma_{71} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \tag{1}$$

As per *EN 1993-2:2006 clause 9.5.3(9)* [5], fatigue damage equivalent factor λ related to 2 x 10^6 cycles for railway bridges is calculated by using the following expression:

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \leq \lambda_{\max} \tag{2}$$

5.2. Cumulative damage method

In order to reflect the most onerous and real traffic spectra, fatigue analysis is performed by considering heavy traffic mix. The steps in cumulative damage method can be summarized as follows:

- Stress history is obtained by performing dynamic analysis for each train type in heavy traffic mix on global finite element model for the time step of 0.01 s.

- Stress Range Spectrum is constructed by separating each stress range spectrum in stress history through rainflow technique [1] method;
- Endurance limit (N_i) of each stress block is calculated by the following relationship

$$N_i = N_c \left[\frac{\Delta\sigma_c / \gamma_{Mf}}{\gamma_{Ff} \Delta\sigma_i} \right]^m \tag{3}$$

- Total damage, D is calculated by summing up the damage caused by individual blocks in the stress histogram

$$D = \sum_i D_i = \sum_i \frac{n_i}{N_i} . \tag{4}$$

5.3. Identification of critical welded detail

Based on the preliminary fatigue analysis, it is observed that the connection between the cross girder and main girder at the center of span is the most critical welded detail, as is shown in Figure 6. Since the λ-coefficient method and cumulative damage method are based on the nominal stresses without multiaxial and mean stress effect, they cannot give a clear picture about actual fatigue damage. Hence, a detailed multiaxial fatigue assessment is needed to get actual damage rate due to each train pass.

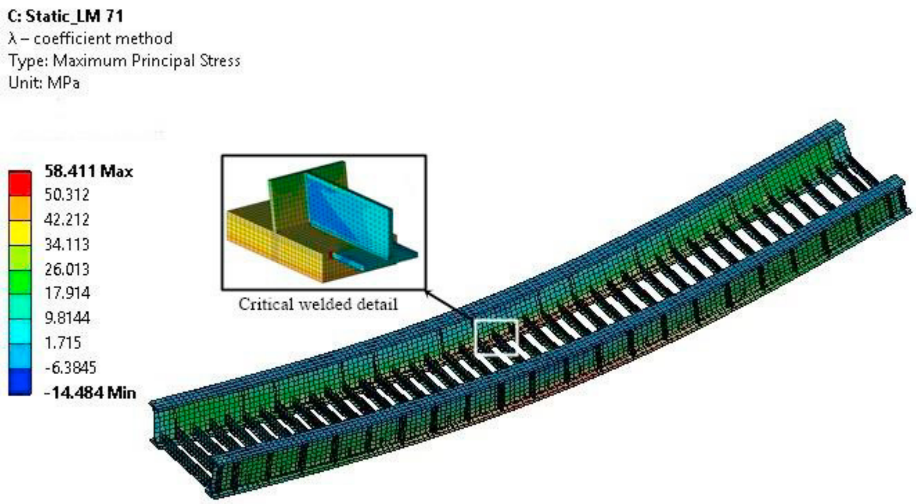


Figure 6a. Critical welded connection between cross girder and main girder at the centre of span

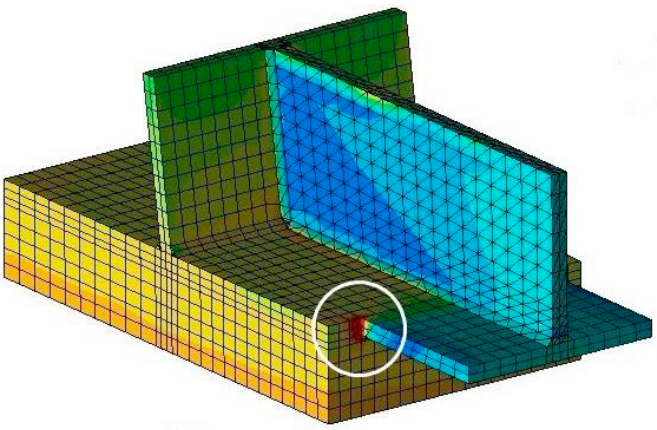


Figure 6b. Stress concentration at flange-to-flange connection in critical welded detail

5.4. Multiaxial fatigue assessment using critical plane approach

The plane on which crack initiates is called critical plane. There is a class of multiaxial fatigue criteria according to which this unique critical plane is the fatigue verification plane where the total damage can be related. It has been found that the uniaxial SN curve based fatigue assessment methods might give non-conservative results for the non-proportional loading. A possible explanation of these results might be due to the fact that the change in principal stress direction activates plastic strains along different slip systems leading to more damage when compared to proportional loading. Therefore the correct fatigue damage is achieved by considering multiaxial non proportional loading.

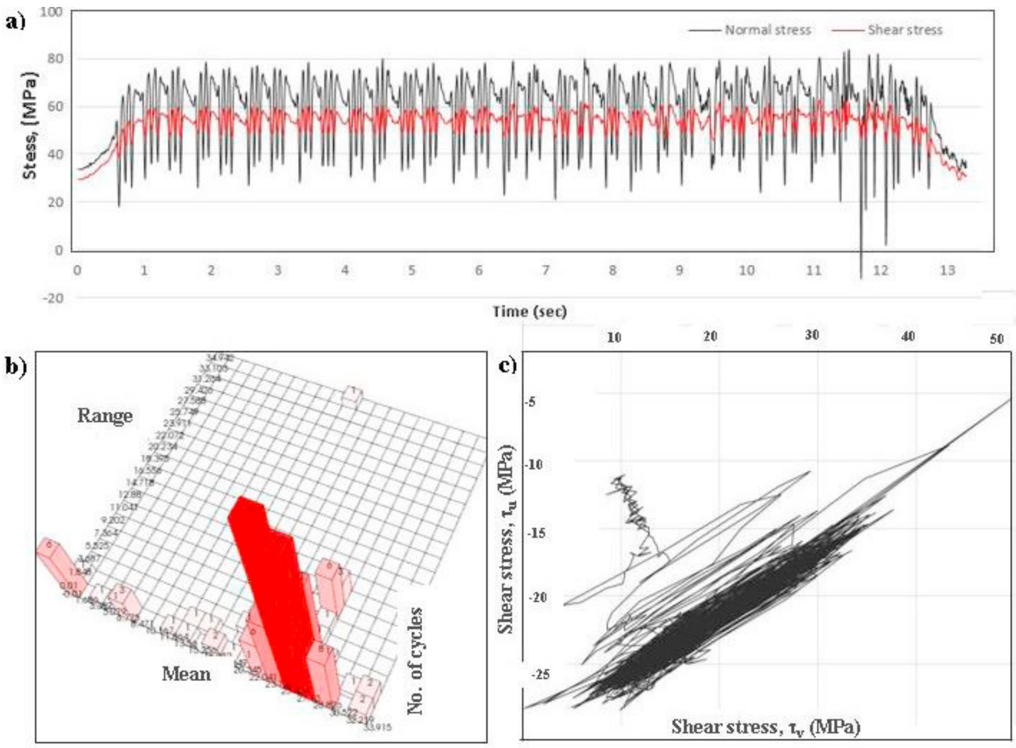


Figure 7. Multiaxial fatigue assessment using C-S criterion for train type 11: a) Normal and shear stress histories on the critical point; b) range mean matrix and c) Shear stress vector path on the critical plane.

The fatigue damage assessment procedured using the C–S criterion can be summarized as follows (see also Figure 7):

1. Critical plane approach based C-S criterion for welded connections is applied in the presence of notches by using the critical point method i.e., by considering stress tensor history at a material dependent critical distance, 0.5L of 0.12mm away from the notch tip where, critical distance L is given by,

$$L = \frac{1}{\pi}\left(\frac{\Delta K_{th}}{\sigma_{af}}\right)^2 \tag{5}$$

2. Mean values of principal stress directions $\hat{1}$ $\hat{2}$ and $\hat{3}$ are calculated from the mean values of the principal Euler angles $\hat{\phi}$ $\hat{\theta}$ and $\hat{\psi}$ by averaging the instantaneous Euler angles $\phi(t)$ $\theta(t)$ and $\psi(t)$ [2, 3, 4] using the appropriate weight function method as given below.

$$\hat{\phi} = \frac{1}{W}\int_0^T \phi(t)W(t)dt \;,\; \hat{\theta} = \frac{1}{W}\int_0^T \theta(t)W(t)dt \;,\; \hat{\psi} = \frac{1}{W}\int_0^T \psi(t)W(t)dt \tag{6}$$

$$W(t) = \begin{cases} 0 & if \quad \sigma_1(t) < c\,\sigma_{af} \\ \left(\dfrac{2\sigma_1(t)}{\sigma_{af}}\right)^{-1/m} & if \quad \sigma_1(t) \geq c\,\sigma_{af} \end{cases} \tag{7}$$

3. The orientation of the critical plane is associated with averaged principal directions and can be determined by the following off-angle between the average direction of the maximum principal stress and the normal to the critical plane,

$$\delta = \frac{3\pi}{8}\left[1 - \left(\frac{\tau_{af,-1}}{\sigma_{af,-1}}\right)^2\right] \tag{8}$$

4. The procedure postulated by Papadopoulos [6] is used to determine mean value and the amplitude of the shear stress acting on the critical plane.
5. For each z-th resolved reversal (obtained from rainflow counting [1] of the normal stress acting on the critical plane), using nonlinear combination of maximum normal stress ($N_{max,\,z}$) and the shear stress ($C_{a,\,z}$) amplitudes acting on critical plane and material parameters taken from the literature (Table 2) equivalent normal stress $\sigma_{aeq,a,z}$ is calculated as follows.

$$\sigma_{eq,a,z} = \sqrt{\left(N_{\max,z}\right)^2 + \left(\frac{\sigma_{af}}{\tau_{af}}\right)^2 \left(C_{a,z}\right)^2} \tag{9}$$

6. Using nonlinear damage rule [4] for $\sigma_{aeq,a,z}$, total damage D at time T_o is obtained as follows:

$$D(T_o) = \sum_{z=1}^{Z} (D_z)^q \tag{10}$$

$$D_z = \begin{cases} \dfrac{1}{2N_o \left(\dfrac{\sigma_{af}}{\sigma_{eq,a,z}} \right)^{-1/m}} & for \quad \sigma_{eq,a,z} \geq c\sigma_{af} \\ 0 & for \quad \sigma_{eq,a,z} \leq c\sigma_{af} \end{cases} \tag{11}$$

$$q = 1 + \frac{0.25}{(1-c)\sigma_{af}} \left(\sigma_{af,a,z} - \sigma_{af} \right) \tag{12}$$

It can be noted that safety coefficient $0 \leq c < 1$and by posing q=1 the damage rule becomes Miner rule.

Table 2. Material parameters of steel S355

Sl.no	Material parameters	Units	Value
1	Ultimate tensile strength of steel, σ_u	MPa	611[1]
2	Fatigue strength under fully reversed normal stress, $\sigma_{af,-1}$	-	276.58[1]
3	Slope of the S–N curve, m	-	-0.15[1]
4	Fatigue strength under fully reversed shear stress, $\tau_{af,-1}$	MPa	183.7[1]
5	Slope m of the S–N curve, m*	-	-0.09[1]
6	Threshold stress intensity, ΔK_{th}	(MPa√m)	7.5[2]

6. Results

The multiaxial fatigue damage due to the heavy traffic spectra on the critical detail (see Figure 6b) is calculated by using critical plane approach based C-S criterion as is shown in Table 3.

Table 3. Multiaxial fatigue damage on critical detail of each train run based on the C-S criterion

Train type	Damage/train	No. of trains/day	Damage
Type 5	2.01E-06	6	0.44
Type 6	1.32E-06	13	0.63
Type 11	1.22E-06	16	0.71
Type 12	1.07E-06	16	0.62
		Total damage, D	2.41

The fatigue damage calculated by using uniaxial SN curve based methods namely λ - coefficient and Cumulative damage are 1.51 and 1.96 respectively. The multiaxial fatigue damage calculated by using C-S criterion is about 2.41. The results and demerits of all three approaches are given and compared in Table 4.

Table 4. Fatigue damage using three different approaches

Method	Damage	Demerits
λ - coefficient	1.51	No dynamic analysis. Instead, Static load model (LM 71) is considered
		Neither variable amplitude nor non-proportionality loading is accounted
		No mean stress correction
		No stress gradient effect
Cumulative damage	1.96	Dynamic analysis using four standardised freight trains as per EN 1993-2 [5]
		Variable amplitude is accounted but not non-proportional loading
		No mean stress correction
		No stress gradient effect
C-S Criterion	2.41	Four standardized heavy freight trains are considered. But, in reality there would be different traffic mix.
		Shear amplitude could be accurately determined using other state of the art methods

It is observed that the fatigue damage of critical welded detail is too high, and therefore, modification in connection/geometry is needed. The modification can be achieved by lifting the cross girder and connecting it to only web part of the main girder as is shown in Figure 8, thus avoiding flange-to-flange connection. The fatigue damage on alternate detail based on the λ - coefficient and Cumulative damage is approximately 0.11.

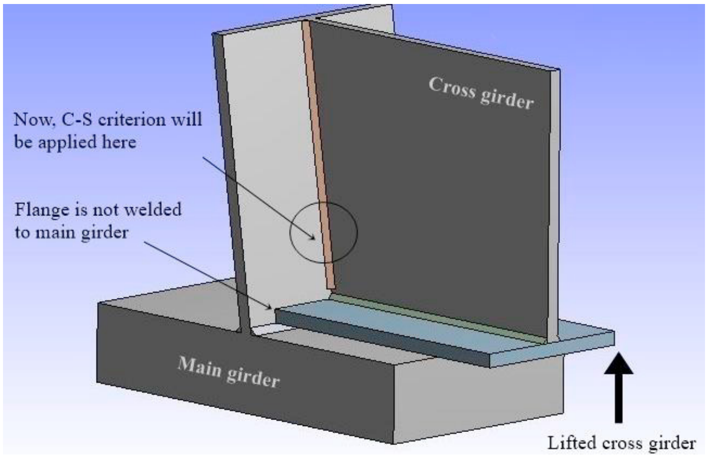


Figure 8. Alternate detail: Cross girder web to main girder web welded connection.

Table 5. Multiaxial fatigue damage on alternate detail of each train run based on the C-S criterion

Train type	Damage/train	No. of trains/day	Damage
Type 5	7.08E-08	6	0.02
Type 6	2.35E-07	13	0.11
Type 11	2.19E-07	16	0.13
Type 12	1.62E-07	16	0.09
		Total Damage, D	0.35

The multiaxial fatigue damage on the alternate detail calculated by using C-S criterion is 0.35 (see Table 5).

7. Conclusions

Accurate and viable fatigue assessment approaches in RAMS system assurance can assist bridge engineers to determine which alternate connections or design modifications are proper for given circumstances. This paper makes an attempt to bring such advancement in RAMS system by proof checking the applicability of C-S criterion on welded connections. Multiaxial notch fatigue is considered by using the point method. It is observed that there are differences in the results obtained by different methods in the calculation of fatigue damage. The multiaxial fatigue assessment using the critical plane approach has led to a higher damage, and the SN curve approach is the method which calculates lesser damage in the critical weld. The reason of this outcome might be due to the fact that the conventional SN curve approach for non-proportional variable amplitude railway traffic loading gives single stress cycle for each train pass. This could disregard other damaging cycles resulting non-conservative values, but further investigation would be needed to strengthen this conclusion. In addition to the demerits mentioned in the Table 4, Eurocode recommends a Miner value of 1 which is the highest possible. The slopes of the Wohler's curves are less steep for higher stress cycles N, as a consequence, lower stress amplitudes are missed out. Furthermore, in the C-S criterion fatigue accumulation due to all small amplitude cycles is accounted. The conclusion is that, the results from the C-S criterion are consistent and applicable to the welded connections in the bridges. Although, more real traffic loading situations need to be processed to fully evaluate the assessment capability of the C-S criterion to welded joints in steel bridges.

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