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FATIGUE ASSESSMENT OF EXISTING BRIDGE MEMBERS

Correct consideration of the service load effects plays a very important role on assessment of railway bridges, especially on fatigue assessment. Eurocode EN 1991-2 introduces twelve types of service trains and prescribes their sequel in normal and heavy duty. The service trains, consisting of locomotive and defined number of wagons, are specified by amount of axle forces and distances between them. However, there is no commentary which would illuminate a background of the service trains technical data. Consequently, it is difficult to say how representative the service load models specified in this standard prescription are in comparison with the actual railway traffic effects, namely in conditions of the Slovak Railways. Therefore, the paper deals with confrontation of the actual railway traffic load effects and the effects of traffic load defined in EN 1991-2. The confrontation is based on fatigue assessment of existing railway bridge truss girder.

1. Introduction

At the present time, the design of structures is connected with consecutive implementing European standards (Eurocodes) into the national standardization. Eurocode EN 1991–2 [1] deals with traffic actions on bridges. For design of railway bridges, this standard introduces load model 71 (LM 71) that may be multiplied by the factor α to get so called classified vertical loads. The classified vertical loads shall be used for design of railway bridge members from the view point of ultimate limit state, especially strength resistance verification. In the case of fatigue resistance verification, the standard [1] establishes twelve types of service trains for modelling railway traffic service load and it also prescribes their sequel in normal and heavy duty. The service trains, consisting of locomotive and defined number of wagons, are specified by amount of axle forces and distances between them. However, there is no commentary that would illuminate a background of the service trains technical data. Consequently, it is difficult to say how representative the service load models specified in this standard are in comparison with the actual railway traffic, namely in conditions of the Slovak Railways.

This paper analyses the effects of service load specified in EN 1991–2 [1] for fatigue assessment of railway bridges in order to compare them with effects of actual service loads on Slovak Railways. The comparison is based on fatigue assessment of chosen fatigue prone bridge structural detail in accordance with the Eurocode 3 [2], [3], by means of linear fatigue-damage accumulation hypothesis according to Palmgren-Miner [4].

2. Analysis of service load response

Actual service load effects of railway bridges may be obtained either by experimental measurement or by numeric simulations. Usually, application of the first approach is financially and time demanding, and therefore it is mostly used for verification of accuracy and calibration of computational model for the second approach.

Such a combined approach was applied to determine actual service load effects on the steel truss riveted girder of the observed railway bridge, which is situated in km 309.309 of railway track

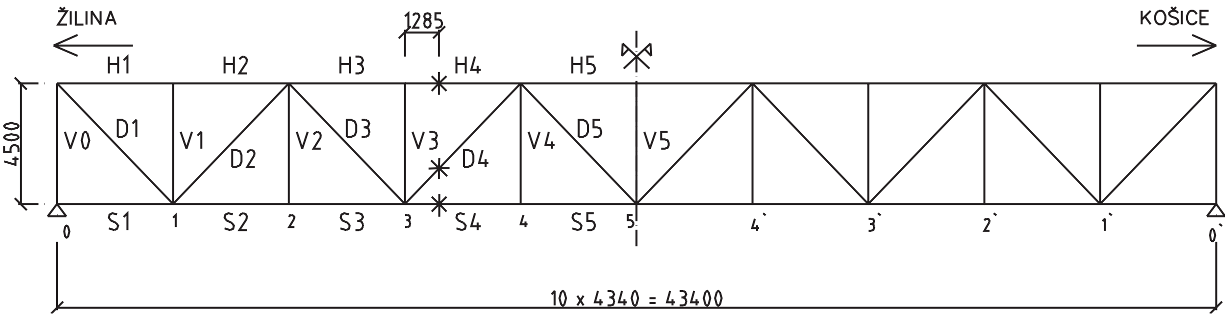

Fig. 1 Longitudinal arrangement of truss girder

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Košice – Žilina. Fig. 1 shows longitudinal arrangement of the truss girder with designation of investigated cross sections of members determining the loading capacity of the girder. Fig. 2 shows a bridge computational spatial model, which was realised using the program IDA NEXIS.

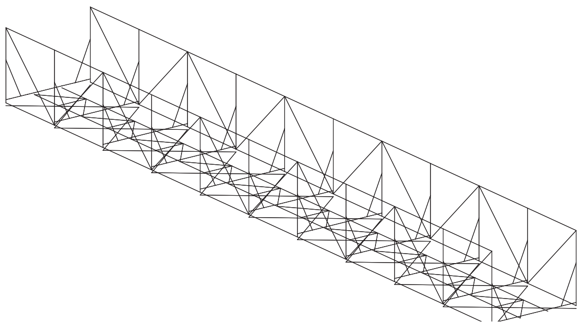

Fig. 2 Spatial computational model of the bridge

Effects of actual railway traffic service load were obtained by simulation of passing 204 freight trains corresponding to actual heavy traffic during one week. Small effects of passenger trains were neglected. Complete data of trains – their composition, frequency, weight and geometry of wagons were acquired from the information system ŽSR IRIS-N. The dynamic effects were taken into account by dynamic factor $\varphi = 1.08$ obtained within numerical simulations in accordance with approach defined in standard [1]. Histograms of stress ranges in the observed cross sections of upper chord, bottom chord and diagonal, respectively, are presented in Fig. 3. The number of single stress-range levels corresponds to considered service life of the bridge $T_d = 100$ years.

Similarly, the effects of heavy railway traffic according to EN 1991–2 [1] were obtained by simulation of passages of freight trains specified in this standard. The standard specifies amounts and spacing of axle-forces of single traffic load models as well as the number of passages per day. The number of passages corresponding to considered service life of 100 years might be obtained by simple multiplying. The histograms of stress ranges in the observed cross sections of upper and bottom chord are presented in Fig. 4.

3 Fatigue assessment of bridge members

The method of fatigue assessment is based on well-known Wöhler's curve of fatigue resistance, which relates the fatigue life to the cyclic-stress range and can be expressed in a logarithmic form as

$$\log N = \log C - m \cdot \log \Delta\sigma$$

where: N is a total number of constant-amplitude-stress cycles to failure, $\Delta\sigma$ is the constant amplitude tensile-stress range, m is the material parameter indicating the constant slope of Wöhler's curve in a logarithmic form and C is the material parameter dependent on the type of notch detail.

To consider the variable-stress range experienced by a fatigue-sensitive bridge component, the direct use of standard Wöhler's curves is not possible. Providing that the stress-range histogram is known, the linear fatigue damage accumulation model according to Miner [4] can be applied to consider the partial fatigue damage at the different stress-range levels. This linear damage accumulation hypothesis, generally known as Palmgren-Miner's hypothesis, can be expressed as follows:

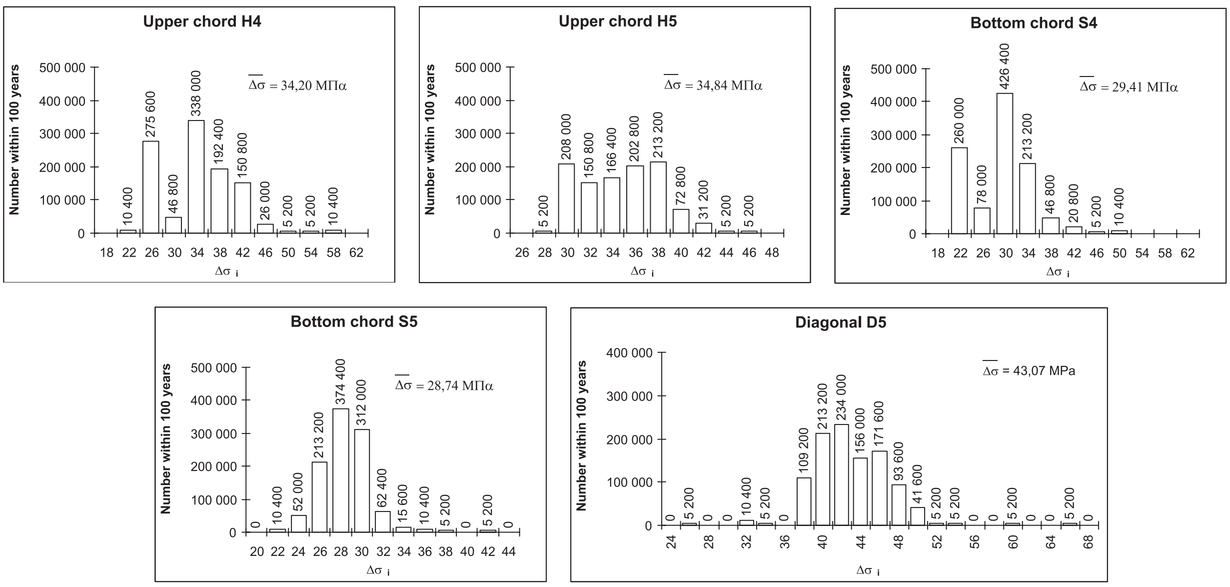

Fig. 3 Histograms of stress ranges due to effects of real heavy traffic within 100 years

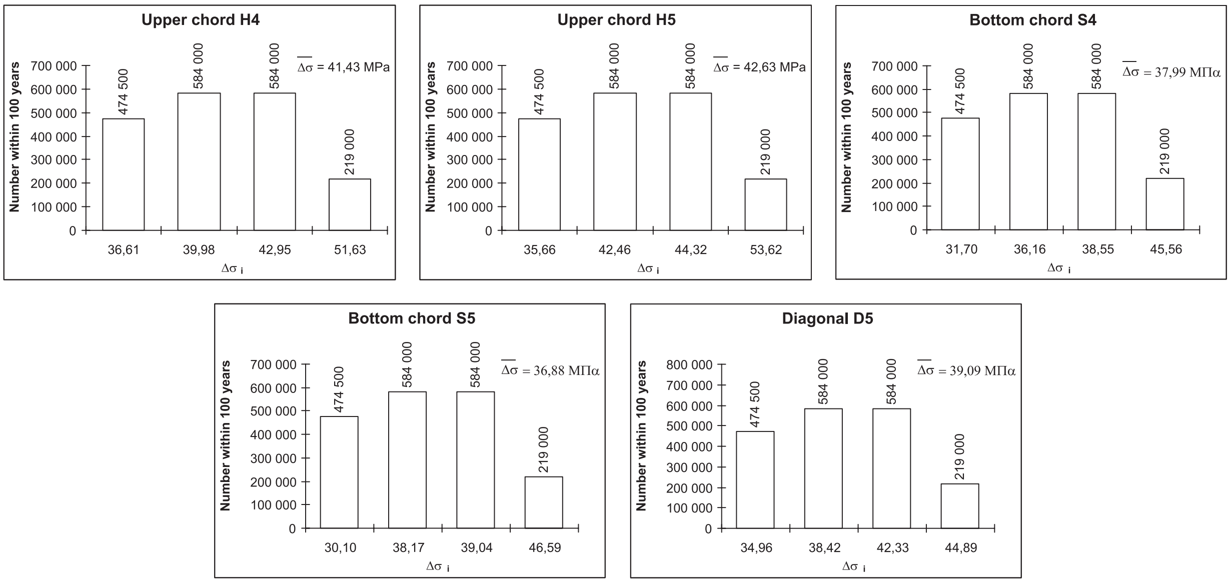

Fig. 4 Histograms of stress ranges due to effects of heavy traffic according to EN 1991-2 within 100 years

$$D = \sum_i \frac{n_i}{N_i} \tag{2}$$

where: n_i is the number of cycles associated with the stress range $\Delta\sigma_i$ for band "i" in the spectrum, N_i is the total number of cycles to failure under constant stress range $\Delta\sigma_i$ and D is the damage accumulation index. The stress ranges below the cut-off limit $\Delta\sigma_L$ (0) are neglected. The failure due to fatigue occurs when $D > 1.0$.

Another way, how the stress-range spectrum may be considered in fatigue assessment, is to replace them by an equivalent constant amplitude stress range spectrum that causes the same fatigue failure as the origin spectrum. It may be expressed again on the basis of Palmgren-Miner's hypothesis using Wöhler's curves. For the assessment purpose, it is useful to relate it to two million cycles, which is the number of cycles corresponding to the fatigue resistance defined in [2], [3]. Then, the equivalent stress range may be obtained from the equation

$$\Delta\sigma_{E,2} = \left(\frac{\Sigma n_i \Delta\sigma_i^{m_1} + \Delta\sigma_D^{(m_1 - m_2)} \cdot \Sigma n_j \Delta\sigma_j^{m_2}}{2 \cdot 10^6} \right)^{\frac{1}{m_1}} \tag{3}$$

where: $m_1(m_2)$ is the slope of the first (second) part of the bilinear Wöhler's curve, n_i (n_j) is the number of cycles associated with stress range level $\Delta\sigma_i$ ($\Delta\sigma_j$) – the index "i" corresponds to the slope m_1, the index "j" corresponds to the slope m_2. Stress ranges under the cut-off limit $\Delta\sigma_L$ (Fig. 5) are neglected.

The standard [2] provides a simplified method for determining the equivalent stress range related to two million cycles, which is defined by relation

$$\Delta\sigma_{E,2} = \lambda \cdot \Phi_2 \cdot \Delta\sigma_p \,, \tag{4}$$

where: $\Delta\sigma_p = |\sigma_{p', \max} - \sigma_{p, \min}|$ is the stress range caused by LM 71, Φ_2 is the dynamic factor and $\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4$ ($\lambda \leq \lambda_{\max} = 1.4$) is the damage equivalence factor considering the damage effect of actual traffic load, the length of the member influence line or area, the traffic volume, the bridge design life, the number of tracks and the fatigue limit. Then, the fatigue assessment is given by relation

$$\gamma_{Ff} \cdot \Delta\sigma_{E2} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}} \,, \tag{5}$$

where: $\Delta\sigma_C$ is the characteristic value of fatigue resistance corresponding to the chosen fatigue prone structural detail according to standard [3], γ_{Ff} is the partial safety factor for fatigue load (recommended value is $\gamma_{Ff} = 1.0$) and γ_{Mf} is the partial safety factor for fatigue resistance depending on the chosen design method (damage-tolerance or safe-life method) and on the consequences of failure according to standard [3].

4. Assessment of fatigue life

Fatigue assessment was realised for an element weakened by a rivet hole and subjected to axial force and bending moment. This detail is classified according to EN 1993-1-9 [3] as the fatigue detail of category 90. Considering the partial safety factor for fatigue resistance $\gamma_{Mf} = 1.15$, the design value of fatigue strength is $\Delta\sigma_C = 90/1.15 = 78.26$ MPa. The Wöhler's curve for the investigated fatigue detail is presented in Fig. 5. The fatigue assessment was realised in two ways:

- using Palmgren-Miner's linear damage accumulation hypothesis – for actual heavy traffic load and for the heavy traffic load according to standard [1],
- using a simplified method of determining the equivalent stress range $\Delta\sigma_{E2}$, to which the total number of cycles to failure N_E under constant stress range $\Delta\sigma_i$ and corresponding cumulated fatigue damage $D = 2 \cdot 10^6/N_E$ were determined.

The results of fatigue assessment of the observed details are presented in Fig. 6.

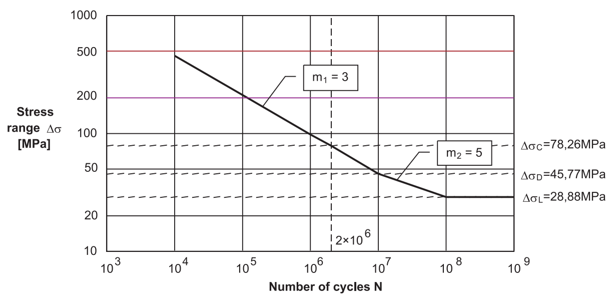

Fig. 5 Wöhler's curve for investigated fatigue detail

5 Conclusions

Based on the results of fatigue assessment it can be stated that all the observed bridge members satisfy the requirement for design fatigue life of 100 years, regardless of the type of service load under consideration. Both the standard approaches for considered railway service load – use of fatigue service trains according to [1] as well as the use of simplified method according to [2], are on the side of safety. Ratios between damage accumulation indexes corresponding to two ways of considering railway service load mentioned above and the damage accumulation index corresponding to actual traffic load are presented in Table 1.

The results of the fatigue assessment show very small damage accumulation indexes of the observed bridge members for all presented approaches. It means that service trains determined for fatigue assessment in the standard [1] do not cause significant fatigue damage of the observed bridge members due to small stress ranges and also due to investigated detail not very prone to fatigue damage. Ratios of damage accumulation indexes emphasise very small stress ranges caused by actual traffic load effects for all the observed bridge members excluding diagonal D5, where the stress ranges due to actual traffic load effects are very close to stress ranges caused by heavy traffic according to standard [1]. The big ratio differences of upper chord H5 and bottom chord S5 result from the smaller stress ranges of these bridge members and lower frequency of the biggest stress ranges compared to bridge members H4 and S4.

At the present time, our research activity is focused on the investigation of the actual railway traffic load effects of the members of the bridge deck. Since the previous experiment was focused only on the main girders, another experimental measurement was necessary. Consequently, the more detailed numerical model was processed using the program IDA NEXIS. At the time, numerical simulations of the actual trains on the observed railway bridge are being realised. These effects will by again confronted with the effects of service load defined in EN 1991-2 [1].

Comparison of the fatigue assessment results — Table 1

	$\frac{D_{\text{traffic load by EC1}}}{D_{\text{actual traffic load}}}$	$\frac{D_{\text{simplified method by EC3}}}{D_{\text{actual traffic load}}}$
H4	3.41	4.38
H5	5.50	9.67
S4	8.14	9.48
S5	33.21	73.21
D5	1.12	1.39

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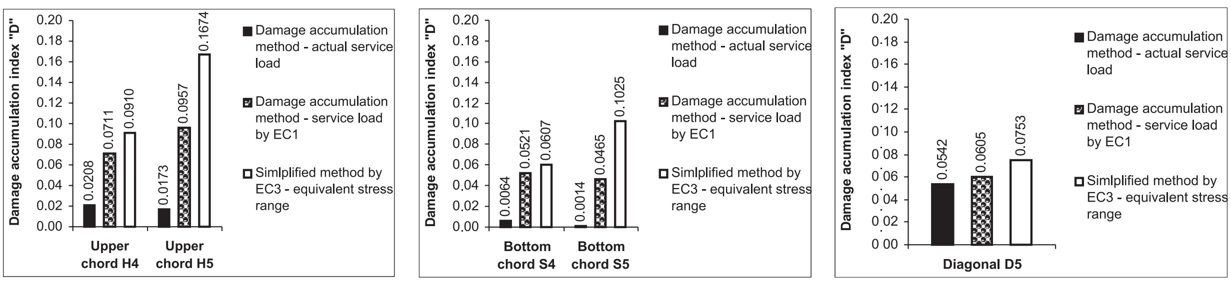

Fig. 6 Comparison of the fatigue assessment results

References

[1] EN 1991-2 *Actions on structures – Part 2: Traffic loads on bridges*, Brussels 2003.
[2] EN 1993-2 *Design of steel structures – Part 2: Steel bridges*, Brussels 2003.
[3] EN 1993-1-9 *Design of steel structures – Part 1.9: Fatigue*, Brussels 2003.
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