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ARTICLE

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The need for new railway bridges is driven by the growing volume of transportation demands for both passenger and freight traffic on railway networks. In the design of these bridges, the fatigue limit state is a criterion that usually limits the allowable applied load level and thus also the utilization of the high strength of the steel material. Therefore, improving the fatigue performance of welded details by high-frequency mechanical impact (HFMI) treatment leads to a more efficient design. However, the fatigue performance of HFMI-treated welds is known to be affected by the mean stress and this needs to be considered in the design of treated welded details in steel bridges. This is rather straightforward if the bridge is subjected to cycles from one type of train but becomes cumbersome when several different sets of trains (e.g. axle loads, axle distances) cross the bridge. In this article, a factor to take the mean stress effect (including self-weight and traffic load variations) into account is derived from traffic data measured in Sweden. Moreover, the mean stress effect is also predicted using the different fatigue load models in the Eurocode. These models either consist of one-load patterns such as LM71, SW/0, and SW/2 or are composed of different trains with different combinations. It was found that the mean stress effect is underestimated by the first group of models. On the other hand, the mean stress predicted by the light traffic mix is found to be close to that calculated using real traffic data, while other mixes (standard and heavy) underestimate the mean stress effect. Therefore, a correction factor to account for the mean stress effects in real traffic is derived (called here λ_{HFMI}). This factor can be used to correct the design stress range for fatigue verification of HFMI-treated welded details in railway bridges.

Keywords fatigue resistance; railway bridge; variable amplitude; mean stress; design; HFMI treatment

1 Introduction

The need for new railway bridges is driven by the growing volume of transportation demands for both passenger and freight traffic on railway networks. In Sweden alone, the Swedish transport administration (Trafikverket) owns more than 4200 railway bridges. In addition to that, Trafikveket has invested more than 25 million USD in constructing new railway bridges in 2009 [1]. Fatigue of welded joints in bridges is one of the design situations that engineers should consider in design. Besides, forensic investigations of collapsed metallic bridges show that fatigue is one of the most frequent failure modes [2]. Therefore, increasing the fatigue resistance of welded details in railway bridges would result in utilizing the material better in the ultimate limit state and thus saving weight and reducing material consumption [3].

For a more efficient design, increasing the fatigue strength of steel welded details is crucial. Therefore, several post-weld treatment methods have been developed in the past decades. Grinding and remelting are two of these methods that improve the geometry by smoothening the transition at the weld toe [4]. However, these methods can be time-consuming and labour-intensive. Another solution for fatigue problems is to avoid welding and use bolting to join steel plates. However, bolting is a relatively expensive metal joining method, particularly on existing structures that need upgrading.

High-frequency mechanical impact (HFMI) is another post-weld treatment method that aims at increasing fatigue resistance by inducing compressive residual stresses at the weld toe. It also decreases the stress concentration and improves the hardness locally. Therefore, HFMI treatment is proven to be one of the most efficient methods for improving fatigue strength in the high-cycle fatigue regime [5], where the loads are of relatively low-stress ranges when compared with the steel strength such as in steel bridges.

To incorporate HFMI treatment in the design of railway bridges, three design aspects should be taken into account. First, higher fatigue strength shall be assigned to the treated welded details. The International Institute of Welding (IIW) assigns different fatigue improvement classes to consider the HFMI effect depending on many aspects such as steel strength, plate thickness, and detail type [5]. Secondly, high stresses that cause relaxation or reduction of residual stresses should be avoided or controlled since HFMI treatment is very reliant on the beneficial-induced compressive residual stress that contributes to fatigue strength improvement. Therefore, the load applied to the steel weldments shall be limited depending on the type of the constructional detail. Based on the results of fatigue testing, different limits have been set for different details [6].

The third design aspect which needs to be considered for HFMI-treated welded details is the mean stress (i.e. the stress ratio) effect. Several studies have demonstrated that the level of fatigue strength improvement decreases as the stress ratio increases [7–10]. This indicates that the stress ratios generated by the traffic need to be evaluated. Alternatively, the stress ratios generated by the used fatigue load model can be used to consider this design aspect (i.e. mean stress effect) if the load model is proven to represent the mean stress effect of real loads. The authors are not aware of any published work to verify this for railway traffic.

The IIW recommendations [5] propose decreasing the fatigue strength class of the HFMI-treated welded details to account for the increased stress ratio. A stepwise penalty from zero to three classes is assigned depending on the stress ratio. This method is applicable when the welded details are subjected to constant amplitude loading where the load cycle defines values of stress range and mean stress. However, this is not the case when different train types pass on railway bridges. In addition, the self-weight stress may increase the mean stresses acting in the bridge welded details [6].

Another drawback of the aforementioned IIW method in considering the stress ratio is the lack of data for stress ratios higher than 0.52. However, several investigations have been made to extend the method to higher R-ratios. Mikkola et al. found that four fatigue strength class re- ductions should be assigned for R-ratios greater than 0.5 [8]. More fatigue tests were conducted under higher R-ratios in [9], and the results supported the trend of the IIW recommendations. Shams-Hakimi et al. [10] suggested the extension of the method to R-ratio ≈ 1. In principle, the trend follows a decrease of one fatigue strength class (i.e. FAT class) per 0.12 increase in the stress ratio. For instance, a welded detail subjected to a loading cycle with a stress ratio of 0.35 should be assigned two fatigue strength classes lower than those assigned by the IIW recommendations, as shown in Fig. 1.

In a previously published article, Shams-Hakimi et al. developed a design method to consider the effects of both the bridge's self-weight and the variation of traffic load on the mean stress in HFMI-treated highway bridge welded details [10]. The method is derived using real traffic data from Sweden and the Netherlands [11]. The method is characterized by its simplicity and appropriateness for the design of steel and steel–concrete composite highway bridges. The mean stress is represented by a factor called λ_{HFMI} which is expressed in Eqs. (1), (2). Φ in these equations takes the self-weight stress into account. Nonetheless, the authors are not aware of any similar work made for steel welded details in railway bridges.

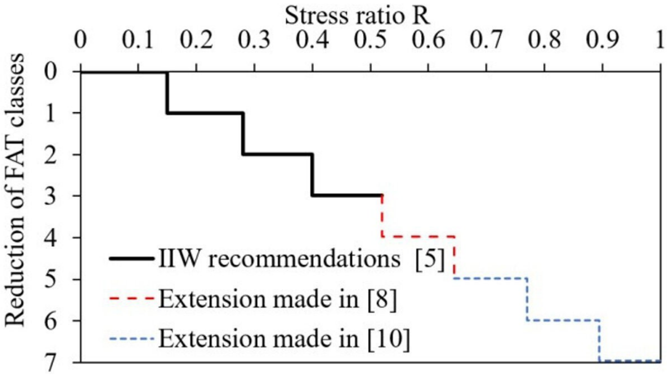

Fig. 1 IIW method to include the mean stress effect in HFMI-treated welded details [5]

$$\lambda_{\mathrm{HFMI}} = \frac{2.38\Phi + 0.64}{\Phi + 0.66},\ \lambda_{\mathrm{HFMI}} \geq 1, \quad \text{for the mid-span} \tag{1}$$

$$\lambda_{\mathrm{HFMI}} = \frac{2.38\Phi + 0.06}{\Phi + 0.40},\ \lambda_{\mathrm{HFMI}} \geq 1, \quad \text{for the mid-support} \tag{2}$$

Some of the railway networks in Sweden are designed to carry the same type of train for their whole design lives, such as 'Malmbanan' in the north of Sweden [12]. The mean stress effect, in this case, is obtained directly by moving the train on the influence line and decreasing the fatigue strength (FAT class) depending on the stress ratio as stated earlier. However, most railway bridges are designed to transport different types of trains. The authors are not aware of any published research on the effect of mean stress on the design of HFMI-treated welded details in railway steel bridges transporting mixed train traffic. Moreover, it is not yet verified that the R-ratios generated by the Eurocode train load models (SW/0, SW/2, LM71, lights standard, and heavy traffic mixes) are representative of those corresponding to real traffic. In this article, train data are collected from measuring stations in Sweden. Then, design equations are derived to take the mean stress due to bridge self-weight and train loads into account. Moreover, the different Eurocode train load models are employed to predict the mean stress effect, so they are compared to the proposed design equations. Moreover, worked examples are presented to show how HFMI treatment is to be incorporated into the design of railway bridges.

2 Methodology

2.1 Traffic measurements

The Swedish transport administration (Trafikverket) uses the equipment 'SCHENCK MultiRail WheelScan' to measure the weight of each axle passing over the bridge. In addition to the axle load, the configuration of axles and the number of axles per train are also recorded. In total, 220 measured trains are used in this article. These trains are different in terms of the axle load, spacing between axles, and the number of wagons. The cumulative distribution of the axle loads and axle distances of the studied data pool is given in Fig. 2.

2.2 Framework for predicting the mean stress effect

Matlab functions are built to process the train data, simulate traffic on bridges, and run the trains over the bending influence lines. Influence lines for different locations on bridges with various span lengths (100 m ≥ L ≥ 5 m) are studied. Simply supported and symmetrical double-spanned continuous bridges with constant bending stiffness are considered. Five locations on the continuous bridge are studied starting from the middle of one of the spans to over the intermediate support, while only the mid-span is considered for the simply supported bridge. In total, 60 analyses were run. The influence lines of the different studied positions are shown in Fig. 3. The lines in the

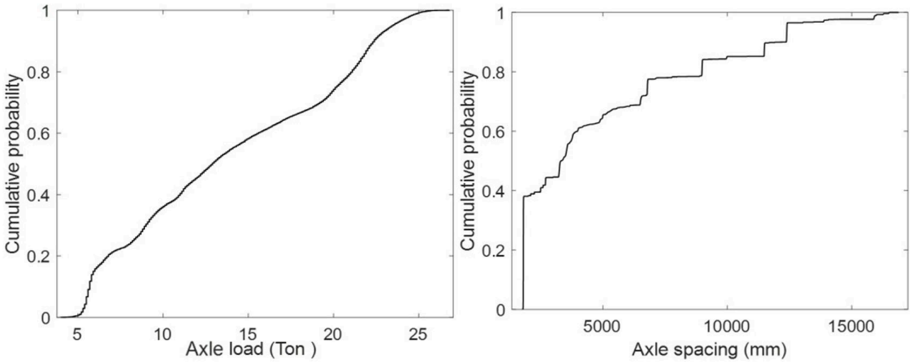

Fig. 2 Axle load and axle distance distributions of the measured trains

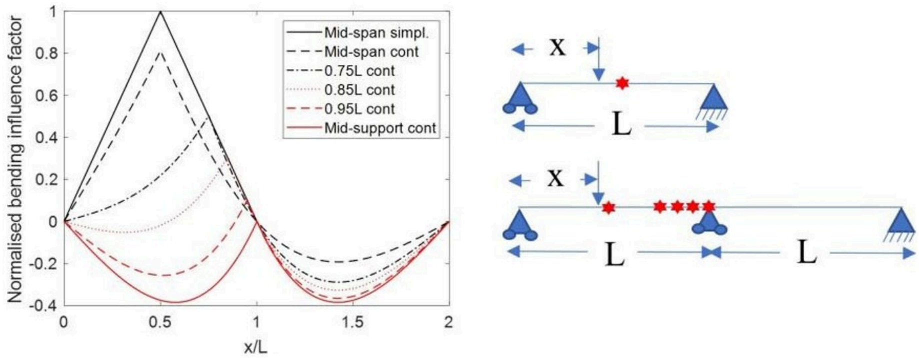

Fig. 3 bending influence line factors

figure are normalized to the maximum obtained bending influence factor (which corresponds to the mid-span in a simply supported bridge).

As mentioned before, the design method should consider the variation of the stress ratios due to both train loads and the bridge's self-weight. The former can be achieved by producing different influence lines with different spans and running the measured trains over these lines. The mid-span of a simply supported bridge and five different locations in double-span continuous bridges is studied. Constant bending stiffness is assumed for all studied cases. The generated load cycles are then identified using the rainflow counting method, and the equivalent stress range ΔS_{eq} is then calculated as given in Eq. (4). Moreover, a magnification factor f to account for the decrease in fatigue strength with increasing stress ratios is defined. This factor is used to magnify the stress range and incorporated in the adjusted equivalent stress range ΔS_{eqR} presented in Eq. (5). The stress ratio effect λ_{HFMI} is then calculated as the ratio between ΔS_{eqR} and ΔS_{eq}, as shown in Eq. (6). Besides, the self-weight effect is considered via the defined variable called Φ as given in Eq. (7). Finally, λ_{HFMI} is plotted against an interval of Φ values from 0 to 9.

$$f_i = 0.5R_i^2 + 0.95R_i + 0.9,\ f_i \geq 1 \tag{3}$$

$$\Delta S_{eq} = \left(\frac{\sum (n_i \Delta S_i)^m}{\sum n_i}\right)^{\frac{1}{m}} \tag{4}$$

$$\Delta S_{eqR} = \left(\frac{\sum (n_i(\Delta S_i f_i))^m}{\sum n_i}\right)^{\frac{1}{m}} \tag{5}$$

$$\lambda_{HFMI} = \frac{\Delta S_{eqR}}{\Delta S_{eq}} \tag{6}$$

$$\Phi = \frac{S_{SW}}{\Delta S_{max}} \tag{7}$$

2.3 Fatigue load models

Different load models for fatigue verification of railway bridges are given in Eurocode 1, part 2 [13]. These models were calibrated using data measured decades before the data in this article [3]. Therefore, their capability of predicting the mean stress effect is questionable. Some of these models consist of a single load pattern such as LM71, SW/0, and SW/2. LM71 consists of both concentrated and distributed loads. The load arrangement of the model is shown in Fig. 4. The load should be positioned in a way to give the maximum and minimum action load effects (i. e. bending moment). Besides, designers are allowed to use SW/0 or SW/2 for heavy traffic passing over continuous bridges, the model consists of only a uniform load distributed over a specific distance, as shown in Fig. 4. It is noteworthy that these models can be used for the design of railway bridges subjected to different types of trains though the models consist of one load pattern. These load models are to be used for fatigue verification in conjunction with the λ coefficient method [3, 13].

As an alternative, Eurocode allows combining several standard freight, passengers, and high-speed trains composed of several wagons with specified axle loads and configurations [13]. Depending on the volume of traffic on the network, one of three

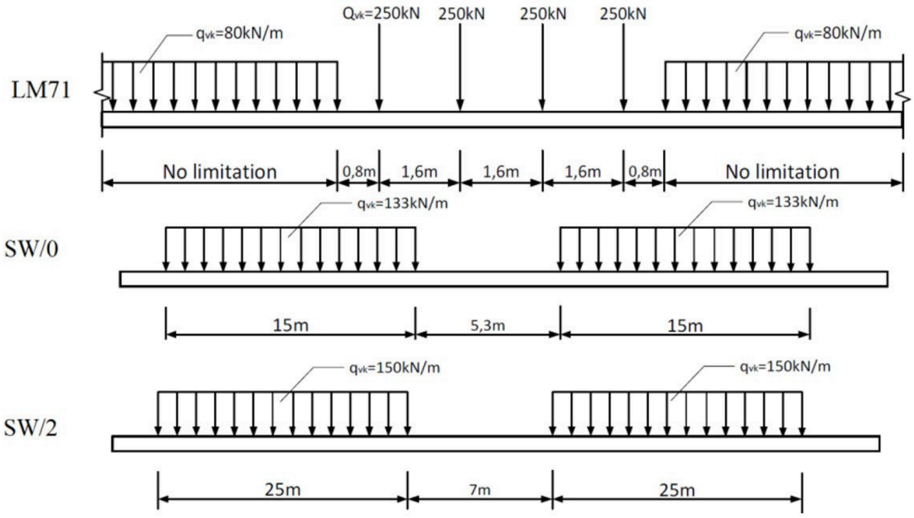

Fig. 4 Fatigue load models LM71, SW/0, and SW/2, respectively (adopted from [13])

predefined mixes can be selected: light, standard and heavy mixes. The number of daily trains passing for each mix is defined in Tab. 1. Eurocode allows the local traffic authorities to use other traffic mixes to better represent rail traffic. For example, in Sweden, Trafikverket defines another mix which is similar to the standard mix but with different freight trains [3]. The number of trains in each 'traffic mix' together with the train types is given in Tab. 1. In addition, Trafikverket suggests the use of model 13S for 'Malmbanan' railway line, Fig. 5, which shows this load model.

For the different traffic mixes, the mean stress effect can be estimated using Eqs. (3)–(6) as they consist of several trains. In other words, it is evaluated the same way as real traffic. On the other hand, LM71, SW/0, and SW/2 generate only one load cycle with one value of mean stress. Therefore, the correction factor f (given in Eq. (3)) is used to reflect the mean stress effect generated by these models.

3 Results and discussion

3.1 Mean stress effect predicted using measured data

All the Φ–λ_{HFMI} generated curves are shown in Fig. 6. These curves correspond to different span lengths (i. e. ranging from 5 to 100 m) and different positions along the bridge (i. e. from the mid-span to over the support in continuous bridges and over the mid-span for simply supported bridges). The mean stress effect is considered in ΔS_{eqR} definition (which considers the stress ratio by magnifying the generated stress ranges via the factor f defined in Eq. (3)).

The highest curve depicted in the figure in a black solid line corresponds to the mid-span section of the simply supported bridge with a span length = 10 m. On the other hand, the curve corresponding to the moment over the middle support is the lowest and is depicted by the red solid line shown in the figure.

Tab. 1 Number of trains per day in each studied traffic mix [13]

	Number of trains [day]												
Train number	1	2	3	4	5	6	7	8	9	10	11	12	Total number of trains [day]
Train type	P	P	HS	HS	F	F	F	F	S	U	F	F	
Standard mix	12	12	5	5	7	12	8	8	–	–	–	–	67
Light mix	10	5	–	–	2	–	–	–	190	–	–	–	51
Heavy mix	–	–	–	–	6	13	16	16	–	–	–	–	67
Traffikverket mix	12	12	5	5	7	12	–	–	–	–	7	9	66

P: Locomotive-hauled passenger train, HS: High-speed passenger train; F: Locomotive-hauled freight train, S: Suburban multiple unit train, U: Underground

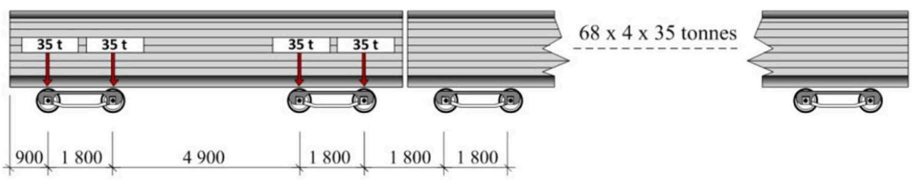

Fig. 5 Train load model 13S (distances are given in mm) [3]

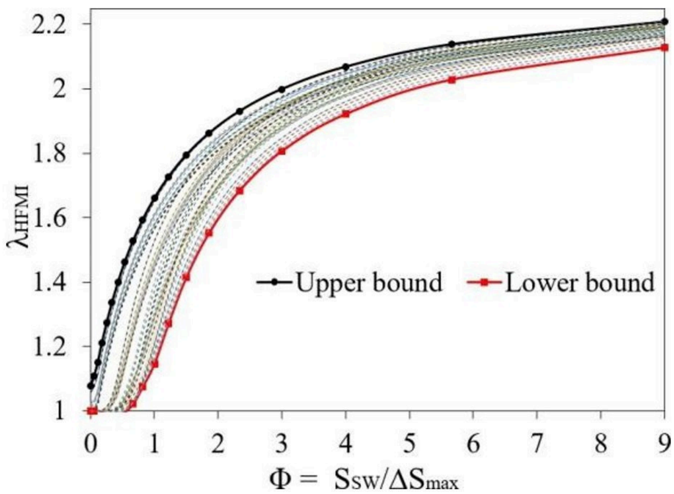

Fig. 6 All the generated Φ–λHFMI curves

The highest Φ–λ_{HFMI} curves corresponding to each studied position along the influence lines are depicted in Fig. 7. The highest curves correspond to the mid-span of the simply supported case, followed by the mid-span of the continuous case. The curve decreases as the location becomes closer to the mid-support (i. e. in the same order shown in Fig. 3). For simplicity, the highest curves corresponding to the mid-span (0.5 L Simply.-Supp), and to the mid-support (0.85 L cont), are proposed for design purposes. The fit expressions for these two curves are given in Eqs. (8), (9). If the expressions yield λ_{HFMI} less than 1.0, it should be replaced with 1.0.

$$\lambda_{\text{HFMI}} = \frac{2.375\Phi + 1.183}{\Phi + 1.074},\ \lambda_{\text{HFMI}} \geq 1, \text{ for the mid} - \text{span} \tag{8}$$

$$\lambda_{\text{HFMI}} = \frac{2.564\Phi + 1.116}{\Phi + 1.608},\ \lambda_{\text{HFMI}} \geq 1, \text{ for the mid} - \text{support} \tag{9}$$

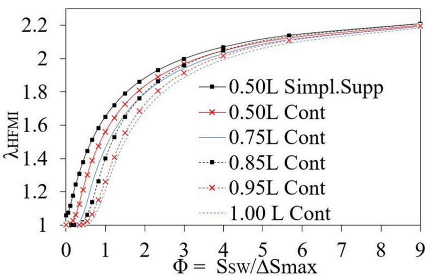

Fig. 7 Φ–λHFMI curves for different bridge locations

Figure 8 shows the Φ–λ_{HFMI} curves for different span lengths. The highest obtained curves correspond to a span length of 20 m for the mid-span section and a span length of 10 m for the mid-support section. The lowest curves for both sections correspond to the longest studied span length ($L = 100$ m). This is because the passage of the measured train on a relatively short-spanned bridge generates both a primary cycle (which has the highest range among other cycles) and other cycles which have lower stress ranges and high mean stresses (i. e. called here secondary cycles). On the contrary, fewer secondary cycles are generated when the bridges are long in relation to the train size, see Fig. 9, which compares the loading cycles generated by the train passage on long and short influence lines.

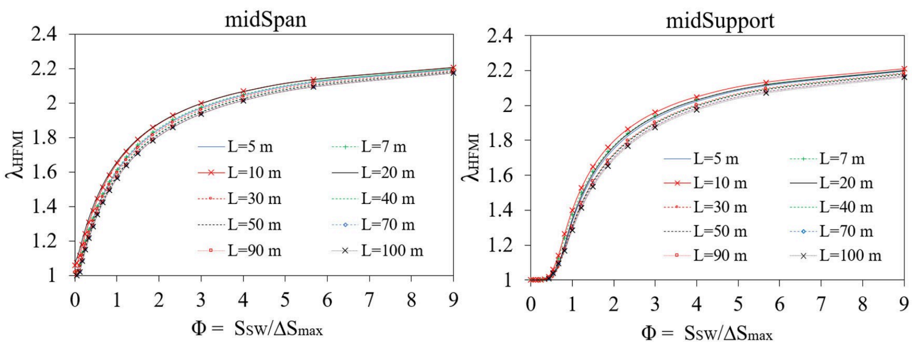

Fig. 8 Φ–λHFMI curves for different span lengths

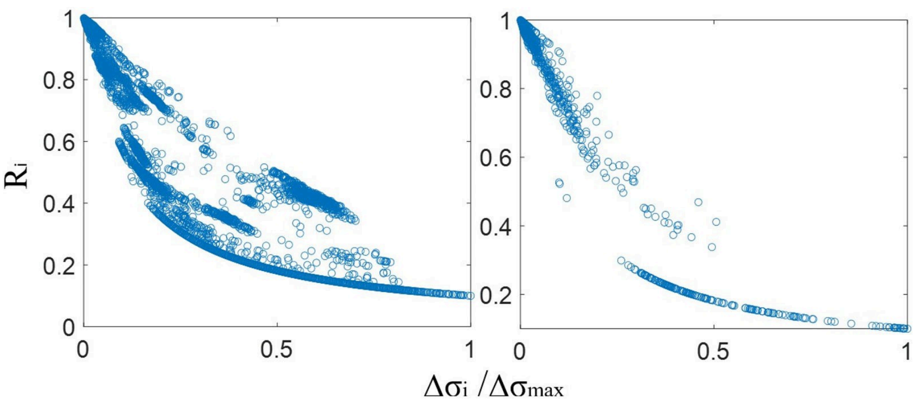

Fig. 9 Generated cycles for different span lengths. Left: 10 m, Right: 80 m

The maximum stress ranges from traffic, ΔS_{max}, are needed to calculate Φ, as shown in Equation (7). However, this parameter is seldom available for designers in the design phase. Therefore, a relationship is suggested between ΔS_{max} and another easy-to-obtain parameter such as the stress range generated by LM71, ΔS_{LM71}, or the maximum stress range generated by any of the traffic mixes, $\Delta S_{max,mix}$, see Fig. 10.

Two solid horizontal lines are plotted to represent the best-fit ratios of the real measured traffic to parameters obtained from the load model (e. g. ΔS_{LM71} or $\Delta S_{max,mix}$). In other words, these lines establish a relationship between the real measured traffic (which is often unknown to designers) to the load model (which can be easily obtained for design purposes). A ratio of 0.73 is selected for LM71. Besides, a higher ratio of 0.9 is chosen for both sections (i. e. midspan and mid-support) for the different traffic mixes ($\Delta S_{max}/\Delta S_{max,mix} \approx 0.9$), as shown in the figure. It is noteworthy that this ratio is attained for all train mixes because train number 5 which generates $\Delta S_{max,mix}$ exists in all mixes (i. e. train 5), as shown in Tab. 1. The maximum error in Φ value due to these approximations does not exceed 10 %, which corresponds to less than 5 % error in λ_{HFMI} value.

The derived methodology introduced in Section 2.2 provides a basis on how to consider the mean stresses when the bridge is designed to carry various types of trains. On the other hand, if the bridge is designed to carry one type of train, the mean stress effect can be obtained directly using Eqs. (3), (5). In other words, the proposed Eqs. (8), (9) are not applicable since they are derived for mixed traffic. An example of such a case is the load model 13S in Sweden [3]. Another example is the 380 measured trains composed of several Fanoo-type wagons on 'Malmbanan' railway line studied in [12]. It can be noted from this measurement that the axle loads are not widely variable when compared to the measured mixed traffic, shown in Fig. 2. The loading cycles generated by the passage of one of these trains on the mid-span of a 10 m-long simply supported bridge are shown in Fig. 11. Herein, the consideration of the mean stress effect is given in Eq. (10) (assuming slope of the S–N curve, m = 5).

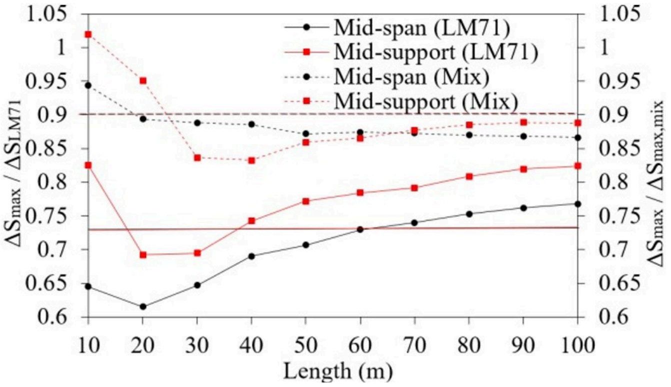

Fig. 10 The ratio of maximum stress ranges from traffic, ΔS_{max}, to the stress range from LM71, Δ*S*LM71, and to the maximum stress range from traffic mix, Δ*S*max,mix

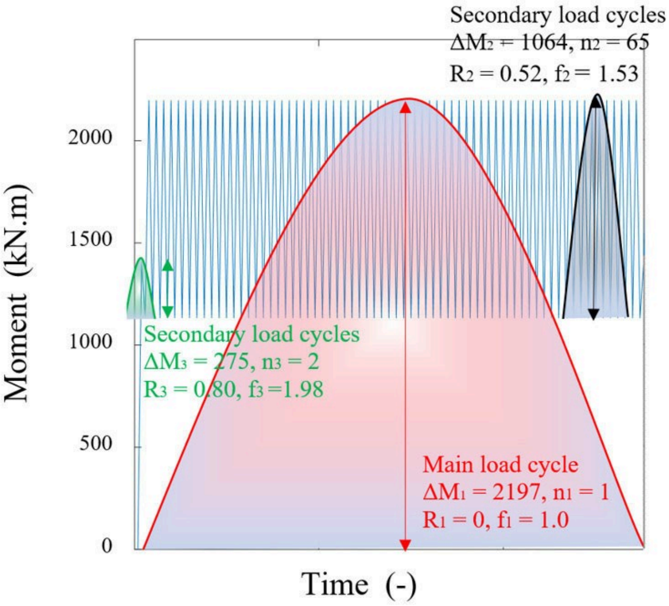

Fig. 11 The time response of the bridge with reference to the bending moment at the mid-span

$$\Delta M_{EqvR} = \left(\frac{(\Delta M_1 \times f_1)^m \times n_1 + (\Delta M_2 \times f_2)^m \times n_2 + (\Delta M_3 \times f_3)^m \times n_3}{n_1 + n_2 + n_3} \right)^{\frac{1}{m}} = 984 \text{ kNm} \quad (10)$$

3.2 Mean stress effect predicted using fatigue load models

In order to investigate whether the different fatigue load models presented in Section 2.3 can predict the mean stress effect generated by the Swedish train traffic, λ_{HFMI} is calculated using these models and compared to λ_{HFMI} calculated in the previous section which corresponds to the real Swedish traffic. For fatigue models consisting of a single pattern such as LM71, λ_{HFMI} is set to be the correction factor (f) given in Eq. (3) (called here λ_{LM71}, $\lambda_{SW/0}$, $\lambda_{SW/2}$). On the other hand, λ_{HFMI} is calculated using Eqs. (4)–(6) for the models that consist of several train types (i. e. traffic mixes) as mentioned earlier.

The comparison of the mean stresses calculated using real traffic versus those predicted via load model is shown in Fig. 12–15. The highest point in these figures is depicted, and the coordinates of these points are shown (x gives the span length, y denotes the self-weight stress normalized to the maximum stress range, and z depicts the ratio of the real mean stress to the mean stress predicted using the load model). Fig. 12 shows that LM71 underestimates the mean stress effect by up to 25 % and 39 % for the mid-span and mid-support sections, respectively. The maximum difference corresponds to short span length and relatively low self-weight in relation to the stress range which is often the case in railway bridges. This clearly shows that the model cannot be used to account for the mean stress in railway bridges. The same observation could be made

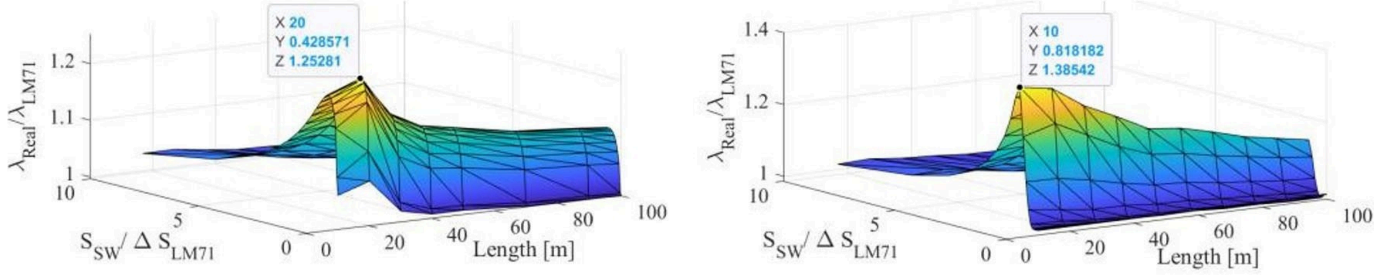

Fig. 12 The ratio of λ_{HFMI} of real traffic and λ_{HFMI} calculated using LM71 for the mid-span and mid-support section respectively

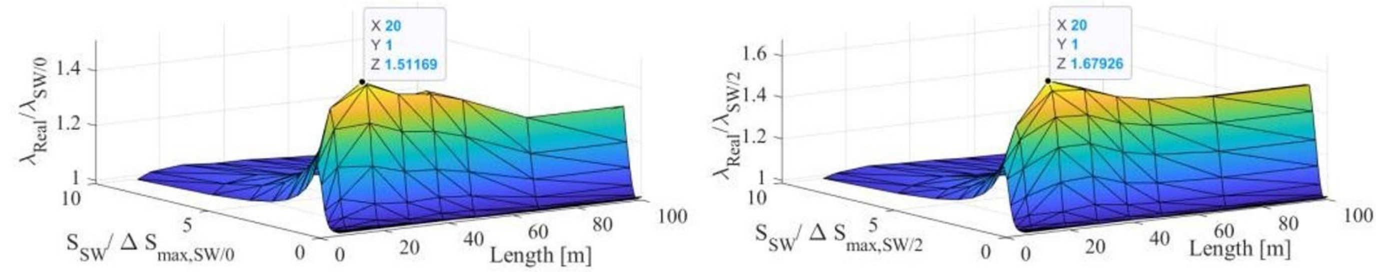

Fig. 13 The ratio of λ_{HFMI} of real traffic and λ_{HFMI} calculated using SW/0 and SW/2, respectively, for the mid-support section

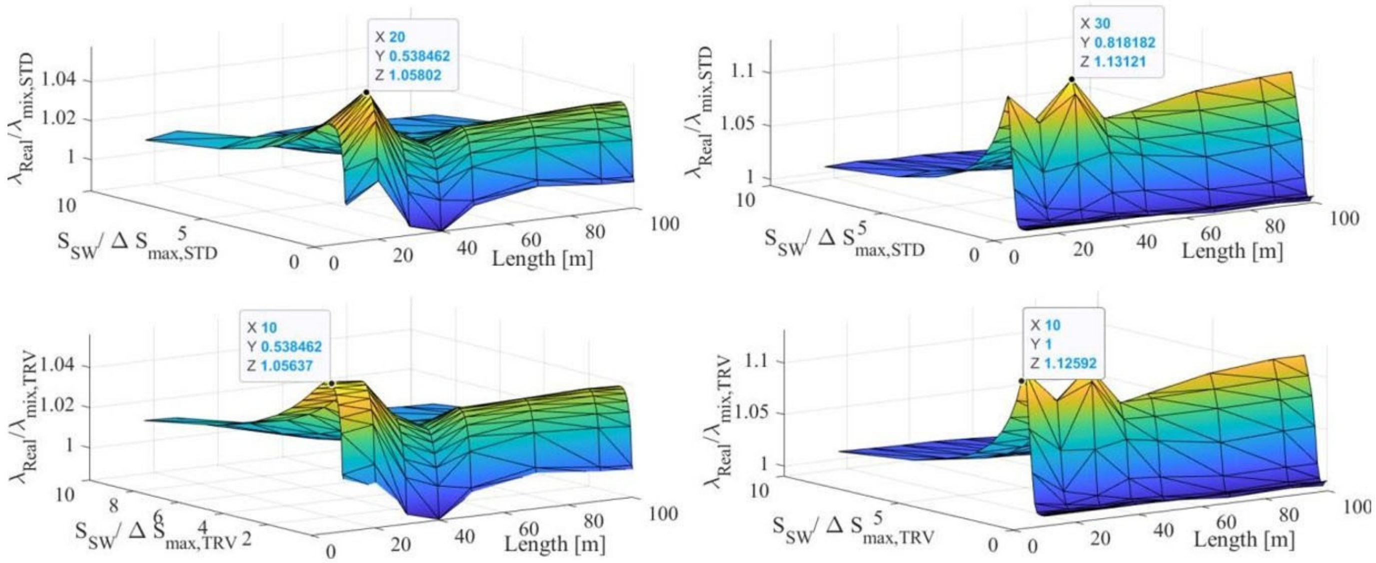

Fig. 14 The ratio of λHFMI of real traffic and λHFMI calculated from standard and Trafikverket mixes for mid-span and mid-support sections respectively

for the rest of the load models with different accuracy levels. Besides, load models SW/0 and SW/2 are the least accurate among all studies models due to the models' heaviness in relation to the measured traffic, see Fig. 13.

The different traffic mixes appear to predict the mean stress effect with better accuracy. The difference becomes even negligible for a span length longer than 20 m for the mid-span section. However, the difference in λ_{HFMI} exceeds 10% in some cases in standard and heavy mixes, as shown in Fig. 14, 15. The Trafikverket-suggested mix gives results close to the standard mix because of the similarity of the considered trains in both mixes, see Tab. 1. On the other hand, the light traffic mix gives the closest λ_{HFMI} to the real measured traffic. This shows that lighter traffic generates larger mean stress as the light trains produce cycles with lower stress range and higher stress ratio, and vice versa, as stated in Section 3.1.

In order to compare the different studied load models in a clearer way, Φ–λ_{HFMI} curves for the different models are plotted and compared to the curve generated by the real traffic. Figure 16 shows this comparison for the mid-span and mid-support sections. In order to unify the scale for all load models, the self-weight is normalized to one parameter(ΔS_{LM71}) regardless of the considered traffic load model. This makes the results slightly different than those in Fig. 12–15. This can be even seen in the definition of Φ in Fig. 16 which is normalized to ΔS_{LM71} regardless of the considered load model contrary to Fig. 13–15. The figure clearly shows that none of the studied models captures the real mean stress effect except the light traffic mix. One possible explanation is that the average R-ratio

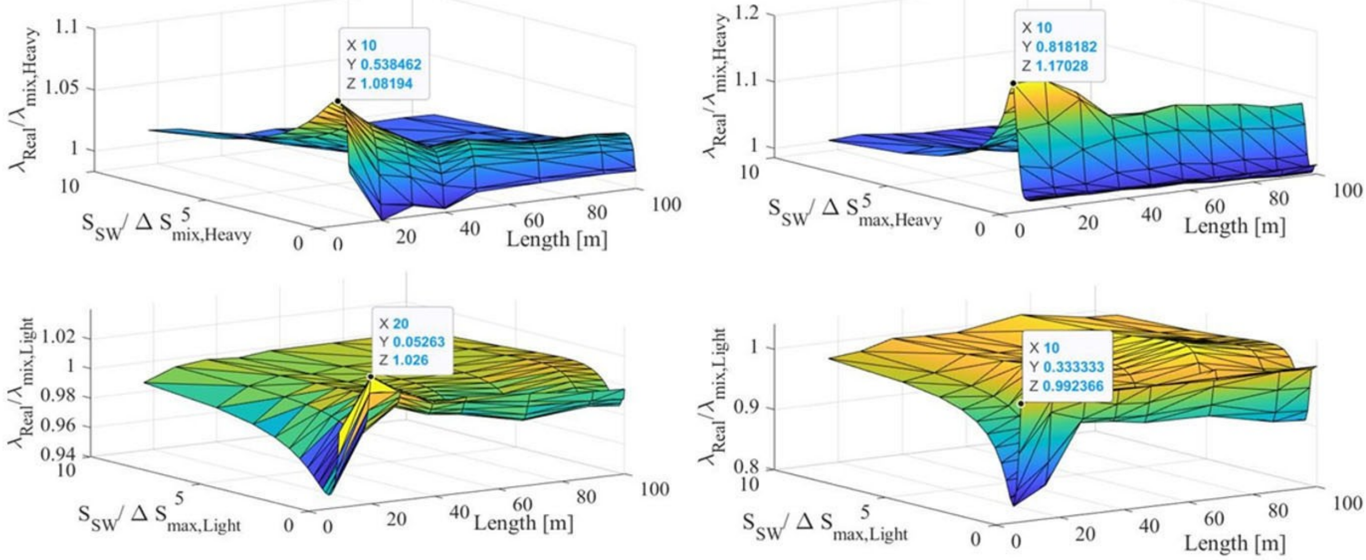

Fig. 15 The ratio of λHFMI of real traffic and λHFMI calculated from heavy and light mixes for mid-support sections respectively

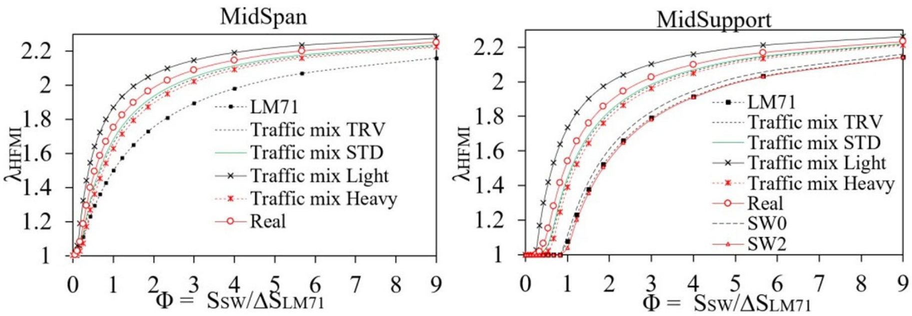

Fig. 16 Comparison of Φ–λHFMI curves for different load models (L = 10 m)

corresponding to the measured traffic is larger than *R*-ratio generated by the load model. This is because the model's generated stress ranges are in general larger than the equivalent stress range of the measured traffic. Based on this, the use of Eqs. (8), (9) appears to be necessary to account for the mean stress effect in railway bridges subjected to mixed traffic.

3.3 Worked example

Fatigue verification of welded details in railway steel bridges is performed using either the simplified λ coefficients method in conjunction with LM71 or the damage accumulation method in conjunction with different traffic mixes or model 13S (the latest is used only in Sweden). In this article, fatigue verification is conducted for an HFMI-treated welded transverse stiffener located in the mid-span of a simply supported bridge. The nominal stress from the self-weight is calculated to be 10.8 MPa. The cross section of the bridge is composed of two steel girders with a common upper flange forming together an open hat-shaped profile, as shown in Fig. 17. The studied welded detail is depicted in the figure. The geometrical cross-section constants of the bridge are given in Tab. 2.

The girders are made of S335 structural steel. The fatigue strength of the HFMI-treated details, $\Delta_{\sigma C,HFMI}$, is known to be

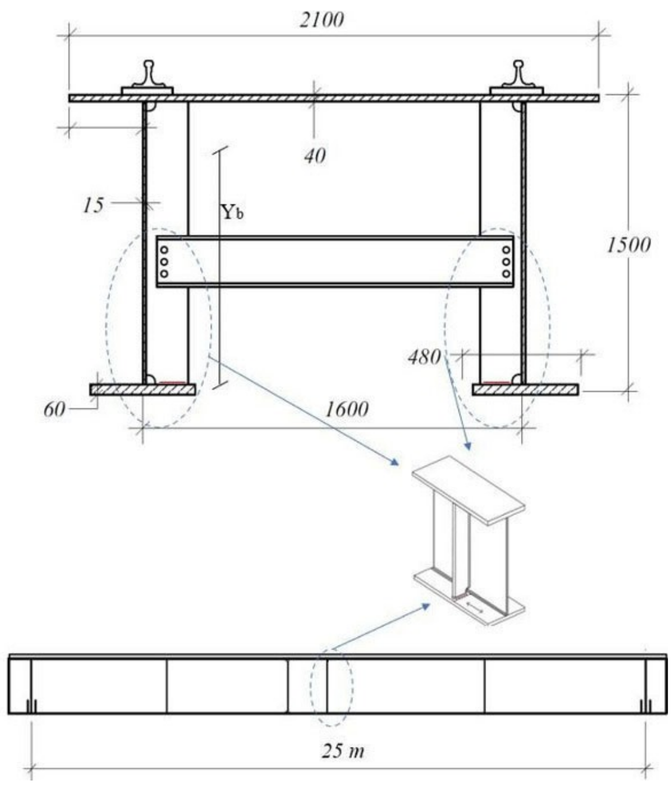

Fig. 17 Top: The railway bridge girder. Bottom: Elevation view of the bridge [4]

Tab. 2 The geometrical cross-sectional constant of the studied bridge

Geometrical parameter	Area	Second moment of area	Section modulus (Bot)	Centroid
Unit	A [m²]	I [m⁴]	W [m³]	Yb [m]
Value	0.1626	0.0753	0.08023	0.813

dependent on the steel grade, as mentioned in Section 1. $\Delta_{\sigma C,HFMI} = 140$ MPa for the given steel [5]. A bilinear fatigue endurance curve (i.e. S–N curve) is used for fatigue assessment, as shown in Fig. 18. No cutoff limit is assigned for S–N curves given in the IIW recommendations [5]. The design life is set to be 120 years. Safe life assessment method with a low consequence of failure ($\gamma_{Mf} = 1.15$ and $\gamma_{Ff} = 1.0$) [14] and a traffic volume of 25 million tons per year are assumed. Herein, the mean stress effect is introduced in fatigue verification using the proposed factor λ_{HFMI}. Only LM71, 13S, and traffic mixes are investigated in this section. SW/0 and SW/2 are not studied because they are only to be used for continuous bridges as stated earlier.

3.3.1 λ coefficients method

The stress range ΔSLM71 is obtained by moving the load model LM71 along the span. The maximum obtained bending moment is then divided by the section modulus to calculate ΔSLM71. Afterwards, ΔSLM71 is to be multiplied by the damage equivalent factor λ and the dynamic amplification factor ϕ, see Eqs. (11), (12). The values of λ_1, λ_2, λ_3, λ_4, and ϕ are obtained from the Eurocode 3, part 2 [13], and given in Tab. 3. Φ, which is needed for λ_{HFMI} calculations, is estimated using the approximation made in Fig. 10 for the mid-span section ($\Delta S_{max} \approx 0.73\Delta SLM71$), see Eqs. (13), (14). The fatigue damage is then calculated according to Eq. 15 using a slope of the S–N curve, m = 5.

$$\lambda = \lambda_1\lambda_2\lambda_3\lambda_4 < \lambda_{max} = 0.68 < 1.36 \tag{11}$$

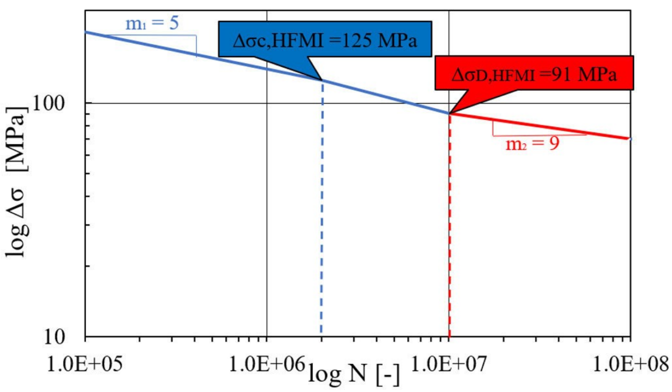

Fig. 18 Fatigue–endurance curve of the HFMI-treated detail (Adopted from the IIW recommendations [5])

Tab. 3 The values of parameters used in λ coefficients method, obtained from the Eurocode [13]

Parameter	λ1	λ2	λ3	λ4	λmax	φ
Value	0.65	1.0	1.04	1.0	1.38	1.157

$$\Delta\sigma_E = \lambda\phi\Delta S_{LM1} = 0.68 \times 1.157 \times 98.3 = 77 \text{ MPa} \tag{12}$$

$$\Phi = \frac{S_{SW}}{\Delta S_{max}} \approx \frac{S_{SW}}{0.73 \times \Delta S_{LM71}} = \frac{10.8}{0.73 \times 98.3} = 0.15 \tag{13}$$

$$\lambda_{HFMI} = \frac{2.375\Phi + 1.183}{\Phi + 1.164} = 1.171 \tag{14}$$

$$D = \left(\frac{\gamma_{Ff}\lambda_{HFMI}\Delta\sigma_E}{\frac{\Delta\sigma_{c,HFMI}}{\gamma_{Mf}}}\right)^m = 0.39 \tag{15}$$

3.3.2 Damage accumulation method

Alternatively, fatigue verification can be made via the damage accumulation method using the Eurocode's train mixes given in Tab. 1. The same fatigue strength and partial safety factors used in the previous subsection are to be used here. The stress time response of these trains including self-weight is shown in Fig. 19. The stress range is calculated using the rainflow counting method. It is noteworthy that all trains generate several cycles with different amplitudes. λ_{HFMI} can be used to account for the mean stress in endurance calculations. Φ is calculated using the ratio given for $\Delta S_{max,mix}$ shown in Fig. 10. Eqs. (16), (17) give the equivalent stress range and the endurance, respectively. indicesi &j give the load cycles with stress range higher and lower than the fatigue resistance $\Delta\sigma_{D,HFMI}$, respectively. $\Delta\sigma_{D,HFMI}$ is the fatigue resistance at the knee point of the S–N curve shown in Fig. 18. m_1 and m_2 are the slopes of the S–N curve given in the figure. The fatigue damage sum can then be calculated from the Palmgren-damage sum rule, see Eq. 18. The fatigue damage is calculated for different mixes in Tab. 4.

Tab. 4 Fatigue damage for different traffic mixes

Traffic type	Fatigue damage
Heavy	0.140
Light	0.025
Standard	0.119
Trafikverket	0.117

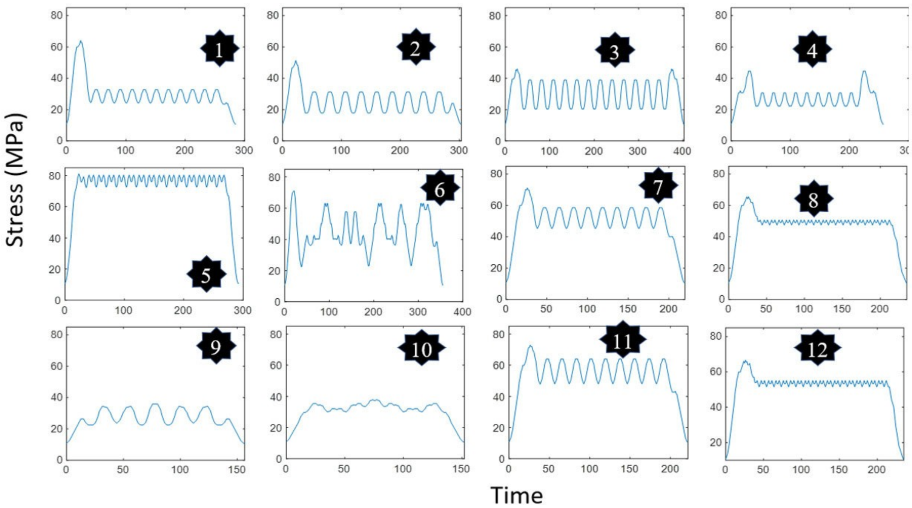

Fig. 19 The time response of the bridge with reference to the bending stresses at the studied detail due to passage of the standard trains (given in Tab. 1)

$$\Delta\sigma_{eq} = \begin{cases} \left(\dfrac{\sum\left(\Delta\sigma_{i1}^{\,m}\times n_i\right)+\left(\frac{\Delta\sigma_{D,HFMI}}{\gamma_{Mf}}\right)^{m_1-m_2}\times\sum\left(\Delta\sigma_i^{m_2}\times n_j\right)}{\sum n_i+\sum n_j}\right)^{m_1}, & \dfrac{\Delta\sigma_{D,HFMI}}{\gamma_{Mf}}\geq\Delta\sigma_{eq} \\ \left(\dfrac{\sum\left(\Delta\sigma_{i1}^{\,m}\times n_i\right)\times\left(\frac{\Delta\sigma_{D,HFMI}}{\gamma_{Mf}}\right)^{m_2-m_1}+\sum\left(\Delta\sigma_i^{m_2}\times n_j\right)}{\sum n_i+\sum n_j}\right)^{m_2}, & \dfrac{\Delta\sigma_{D,HFMI}}{\gamma_{Mf}}\geq\Delta\sigma_{eq} \end{cases} \tag{16}$$

$$N_f = 10^7\times\left(\frac{\frac{\Delta\sigma_{D,HFMI}}{\gamma_{Mf}}}{\Delta\sigma_{eq}\gamma_{Ff}\lambda_{HFMI}}\right)^{m_1\ or\ m_2} \tag{17}$$

$$D = \frac{n_i+n_j}{N_f} \tag{18}$$

If the bridge is designed to transport one type of train (such as Malmbanan railway line), load model 13 S should be used. In this case, the mean stress is not to be included via λ_{HFMI} formulae as stated before but explicitly via the R-ratio generated by the model as stated earlier. Figure 20 shows the stress response due to the passage of the train. One large cycle with a stress range of 113 MPa and 68 small cycles with a stress range of 5 MPa are generated. It can be noted that these small cycles might become more significant if the span length was shorter, as shown in Fig. 11. The correction factors for these two types of cycles are 1.0 and 1.75, respectively. The equivalent stresses and the fatigue damage are calculated in Eqs. (19)–(21).

$$\Delta\sigma_{EqR} = \left(\frac{(\Delta\sigma_1 f_1)^{m_1}\times n_1\times\left(\frac{\Delta\sigma_{D,HFMI}}{\gamma_{Mf}}\right)^{m_1-m_2}+(\Delta\sigma_2 f_2)^{m_2}\times n_2}{n_1+n_2}\right)^{\frac{1}{m_2}} = 64.3\ \text{MPa} \tag{19}$$

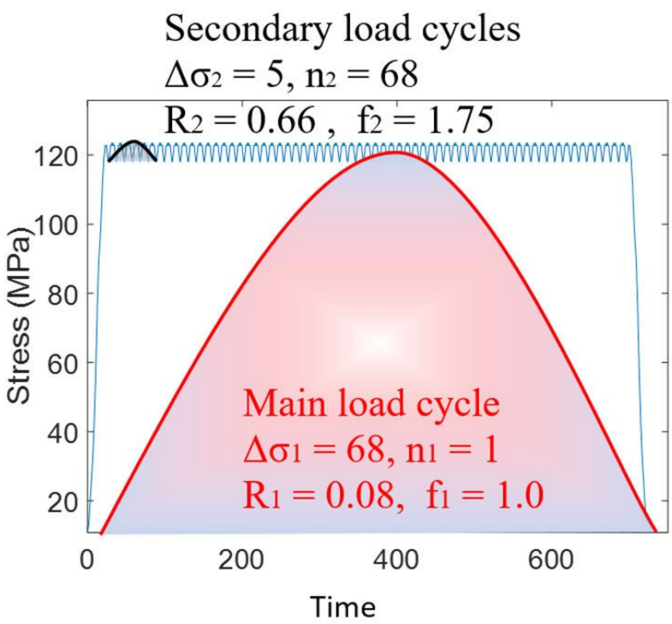

Fig. 20 The stress response in the welded details due to the passage of 13S

$$N_f = 10^7\times\left(\frac{\frac{91}{1.15}}{64.3\times 1}\right)^9 = 6.24\times 10^7 \tag{20}$$

$$D = \frac{4.35\times 10^5+2.85\times 10^7}{6.24\times 10^7} = 0.46 \tag{21}$$

3.4 Limitations and future research

Since only 212 trains were measured from only one country (i. e. Sweden), the results presented in this article regarding the mean stress effect are not necessarily conclusive. More measured stations may yield higher λ_{HFMI} values than those calculated using Eqs. (8), (9). In general, lighter traffic causes a higher mean stress effect, as shown in Fig. 15. Therefore, future research is encouraged to include more traffic data from Sweden and other countries in the analysis to adjust or verify the proposed design equations presented in this article.

If the steel-welded details in the bridge are treated after bridge erection (i. e. after the application of self-weight), the compressive residual stress induced by HFMI treatment is not expected to be affected by the self-weight stresses [10]. Therefore, the self-weight stresses should be put to zero, which makes $\Phi = 0$ in Eqs. (8), (9) (if the bridge is designed to transport mixed traffic) and in the calculation of R ratio (if the bridge is designed to transport only one type of train).

If the mean stress is taken into account, the only remaining design aspect is the maximum ap- plied stresses. Kuhlmann et al. defined the limits at below which the compressive residual stresses induced by HFMI treatment are not expected to relax [15]. The question is which stress design value should be compared to these limits. This issue was studied for highway bridges in [16], and the characteristic load combination is found to be the best studied choice. Railway bridges are less likely to experience the same type of overloads as it is the case for road bridges. Nevertheless, more research is encouraged to study if the characteristic combination can also be used for the maximum stress check in HFMI-treated constructional details in railway bridges.

4 Summary and conclusions

In this article, a method to include the mean stress effect in the design of HFMI-treated welded details in railway steel bridges is introduced. Train data on more than 212 trains are collected and used in traffic simulations using influence lines with different shapes and lengths to assess the mean stress effects that can be expected from real traffic. Moreover, the possibility of introducing the mean stress effect using the different fatigue load models in the Eurocode is investigated. The following conclusions could be drawn.

- The mean stress effect from real traffic can be accounted for using a single parameter designated λ_{HFMI}. Two design equations are derived for calculating λ_{HFMI} for the mid-span and mid-support sections. λ_{HFMI} is found to be larger for shorter bridges because of the emergence of cycles with lower stress range and higher mean stress.
- If the railway bridge is designed to carry only one train type (such as the Malmbanan line in Sweden), the mean stress can be incorporated directly via the R-ratios and the amplification factor f generated by the train passage on the bridge influence line.
- The accuracy of mean stress effect prediction via Eurocodes' fatigue load models is investigated. The models consisting of a single vehicle (LM71, SW/0, or SW/2) is found to be less accurate than those composed of several train types. Eurocode train mixes also failed to conservatively predict the mean stress effect except for light traffic mix. Therefore, the provided expressions for λ_{HFMI} are found to be necessary for incorporating the mean stress effect accurately in design.
- Worked examples are presented to show how the mean stress effects can be accounted for in the design of railway bridges. The design equations in both λ coefficients and damage accumulation methods are adjusted to consider both the change in the S–N curve's slope of the treated detail and the amplification of stress range by λ_{HFMI} to account for the mean stress effect.
- When the steel-welded details are HFMI treated after the bridge erection, the self-weight of the bridge does not affect thee HFMI treatment-induced residual stresses. Therefore, the self-weight should be put to zero in the calculation of mean stresses.
- More research is needed to verify the proposed expressions for λ_{HFMI} using more train data in Sweden and other countries. Moreover, studies on maximum allowable stresses for HFMI-treated welded details in railway bridges are needed.

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