

Fatigue assessment of cope hole details by SHM measurements and multi-level FE models

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ABSTRACT

The fatigue phenomenon is a common issue for steel bridges, affecting their safety and long-term performance. Estimating the fatigue lives of these structures is challenging for bridge managers and owners. This study presents a comprehensive methodology for evaluating fatigue performance of the cope hole detail with two butt weld configurations on the Southern Danube Railway Bridge in Budapest (SDRB). The long-term SHM strain gauge measurement, combined with multi-level finite element models verified and validated through dynamic and static load tests, was utilized for fatigue evaluation. More than 150 FE analyses are performed under various train loads, revealing three critical stress concentration points in the investigated detail. The regression analysis of FE results demonstrates a robust correlation between the measured and the effective stresses experienced at the critical welded detail, allowing for a direct conversion of the stress-time history measured by the SHM strain gauge to the equivalent effective stress-time histories at the relevant critical locations. The nominal stress method and the effective notch stress-based evaluation methods are used for fatigue damage estimation, and the accumulated fatigue damage is assessed to evaluate the fatigue performance over the entire monitoring period. Results indicate that the effective notch stress-based maximum principal stress predicts the shortest lifetime, and the proposed approach is effective for continuous fatigue assessment in steel railway bridges.

1. Introduction

Steel railway bridges are considered one of the backbones and lifelines of trade and transportation. Therefore, bridge safety is a critical task to overcome the inconvenient consequences. The fatigue phenomenon is a common issue for steel bridges, affecting their safety and long-term performance. The estimation of fatigue lives of these structures is challenging for bridge managers and owners (Cai et al., 2017). Fully welded members are commonly used in modern construction, which often requires geometric modifications during the assembly process. Cope holes are one of these modifications used at the intersections of web and flanges to facilitate the welding process, which primarily employs both fillet and butt welds. Due to the manufacturing process, cope holes became a region of stress concentrations, which is a fatigue-critical detail (Cai et al., 2017; Liao et al., 2020a, 2020b; Miki and Tateishi, 1997; Röscher et al., 2023; Śledziewski, 2019).

The fluctuating stresses at the welded joints of steel bridges due to repeated loads make them vulnerable to fatigue deterioration

mechanisms. Although the stress levels are typically below the yield strength they may eventually lead to crack initiation and propagation. Surveys of fatigue damage in steel bridges have shown that more than 90 % of reported damage cases are associated with deformation-induced effects, which are commonly generated by unintended or overlooked interactions between different load-carrying members (Haghani et al., 2012). Field investigations of steel highway bridges indicated that fatigue cracks frequently occur in welded details subjected to secondary or distortion-induced stresses, mainly due to improper structural details and repetitive loading by overloaded vehicles (Nishikawa et al., 1998). More recent reviews highlighted that fatigue in steel bridges is a multi-factorial phenomenon not only affected by traffic load spectra, but also influenced by corrosion, manufacturing quality, and even seismic actions (Milella, 2024; Pipinato et al., 2011).

To overcome the abovementioned challenges, Structural Health Monitoring (SHM) has become a fundamental tool for monitoring the structural behaviour of bridges under service loads. SHM systems consist of various sensor types that continuously measure the structural

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response under actual traffic (Azhar et al., 2024; Deng et al., 2023; Ko and Ni, 2005; Modares and Waksman, 2013). As an example, the long-term SHM of the Tsing Ma Bridge in China enables the derivation of daily stress spectra that combine both daily routine traffic and extreme events (Ye et al., 2012). A recent study reviewed the capability of SHM of steel bridges for damage detection, calibration of finite element (FE) models, and fatigue life assessment (Vagnoli et al., 2018). Although SHM offers several advantages, sensors are often not installed directly at fatigue-sensitive locations, such as cope hole edges or near weld toes, which limits their ability to capture highly localized stress responses at critical details. Therefore, integrating SHM with FE models and refined fatigue evaluation methods becomes an efficient tool for fatigue assessment of welded details (Menghini, 2025).

Fatigue evaluation approaches have improved from simple to more advanced methods. Despite the shortcomings of the nominal stress method, which overlooks the local effects of geometric discontinuities, it remains widely used in design codes for simplicity (Hobbacher, 2008). The hot-spot stress (HSS) method evaluates fatigue by extrapolating stresses calculated from FE models or converted from strain-gauge measurements near the weld toe, accounting for geometric discontinuities while excluding the local weld-toe notch effect (Aygül et al., 2012). Although the HSS method provides a refined stress evaluation through surface-stress extrapolation, uncertainty may remain in the resulting fatigue performance due to the extrapolation procedure and variations in weld quality and local defects (Aygül et al., 2013). Despite their applicability for evaluating fatigue strength, the nominal and hot spot methods are limited by neglecting the notch effect of the weld.

To overcome the identified shortcomings, a more refined effective notch stress method (ENS) was introduced, which applies a fictitious 1 mm radius to the actual weld root or toe geometry to represent the notch effect, and then the notch stress ranges are compared with a single S-N curve (Fricke, 2008). The effective notch stress method, standardized in the IIW recommendations, provides a unified framework that reduces the scatter in fatigue strength of the welded joints (Fricke, 2008; Hobbacher, 2008). Several studies investigated the fatigue assessment of decks and rib-cross girder connections on the orthotropic steel bridges. It is found that the ENS is more reliable compared to the experimental results than the nominal or hot spot methods (Aygül et al., 2012; Huang et al., 2019). Moreover, ENS has been used to evaluate fatigue performance of sensitive details such as cope holes in truss bridges (Cai et al., 2017) and U-rib joints in orthotropic decks (Wang et al., 2023). The escalation of the ENS approach is verified by its formal inclusion in the updated Eurocode 3, which considered it in addition to the nominal and hot spot stress methods for fatigue evaluation (Lukić et al., 2023).

In this context, cope holes are distinctive as one of the most critical details vulnerable to fatigue in the steel railway bridges. Field surveys and structural diagnostics have consistently recognized the cope hole details as a common crack initiation zone (Haghani et al., 2012). Experimental and numerical studies have demonstrated that cope hole geometry, including the hole radius and overall shape, significantly affects the stress concentration factor (SCF), hot-spot stress distribution, and fatigue resistance of welded bridge details (Liao et al., 2017; Ślędziewski, 2019). Experimental studies investigated the fatigue performance of the cope hole detail in the welded joints, demonstrating that the cope holes can be a multi-crack initiation location, inducing complicated fatigue life prediction (Röscher et al., 2023; Wei et al., 2017). Furthermore, an optimization strategy to enhance the fatigue performance of cope hole details was investigated by examining different cope hole schemes, including elliptical holes and cope hole filling plates (Kubiś and Ryjáček, 2017). Studies on large-scale welded joints and orthotropic decks have proven that the ENS is effective for fatigue assessment of cope holes, which can capture local stresses and lead to conservative and realistic predictions (Cai et al., 2017; Wang et al., 2023).

EN 1993-1-9 classifies conventional cope hole welded details as FAT 71 based on the nominal stress in the flange. The IIW recommendations

also classify the cope hole details by considering the combined effect of normal stress in the flange and shear stress in the web at the weld end, the fatigue class decreasing as the shear-to-normal stress ratio increases. In both classifications, the fatigue-critical location is generally associated with the fillet weld toe near the cope hole boundary (British Standards Institution, 2025; Hobbacher, 2008). Although the cope hole joints featuring a butt weld configuration on the flange and web plates are not mentioned in the current design standards, the state of practice in Hungary does utilize this type of connection in most recently constructed railway steel bridges. This configuration has been most notable in the new Southern Danube Railway Bridge (SDRB) in Budapest. However, this detail differs from the standard configuration because the cope hole is located in the intersection region of a combined flange–web butt welded splice, consisting a transverse butt weld in the flange plate and a transverse butt weld in the web plate. This complex local weld geometry may alter the stress field and the actual crack-initiation location; therefore, the standard assumption that the critical point of crack initiation is located at the fillet weld toe may not be directly applicable. While ENS is already standardized by IIW and Eurocode provisions, there remains a significant lack of research in integrating SHM-derived traffic spectra with validated FE + ENS models specifically for cope hole details with a two-butt weld configuration. Addressing this issue offers the potential for more accurate and reliable fatigue life predictions, aligning both with practical monitoring data and the latest methodological standards.

The current study aims to investigate the fatigue performance of cope hole details with two butt weld configurations incorporated in SDRB, integrating finite element modelling techniques and long-term SHM strain gauge measurements to develop a comprehensive methodology for assessing the fatigue performance of critical cope hole details on steel railway bridges. The nominal stress and effective notch stress-based evaluation methods are investigated to establish a precise correlation between the stresses measured by SHM strain gauges and the corresponding effective stresses encountered at the critical detail, thereby supporting the continuous estimation of fatigue damage.

2. The Southern Danube Railway Bridge

2.1. Structural arrangement

The new Southern Danube Railway Bridge (SDRB) of Budapest is a symmetric through-truss structure combined with an orthotropic deck system. The bridge plays a vital role; it carries the heaviest railway traffic in Hungary, connecting the Southern-Eastern and Western European countries. The bridge consists of three separated and identical superstructures, each adjusted on the same pillars, and every individual superstructure carries a single rail, as shown in Fig. 1. The structural members of the bridge are connected by welds, which induce numerous fatigue-sensitive details, such as cope holes, cut-outs, and transverse



Fig. 1. The Southern Danube Railway Bridge in Budapest.

welding in flanges (Mansi et al., 2025a). Fig. 2 illustrates the sensitive details within the typical cross girder of the bridge. A cope hole detail was incorporated at the mid-span of the cross girder contains a complex weld configuration, mainly due to the presence of butt welds in both the flange and web plates, which differs from the standard cope hole details covered by current fatigue design standards and makes it critical for stress concentration and fatigue assessment, as can be seen in Fig. 3. SDRB equipped with the SHM system consists of various sensor types that continuously measure in-service performance and provide valuable measurements (Mansi and Dunai, 2025; Mansi et al., 2025b).

2.2. Dynamic load test of the bridge

The load test of the bridge was executed by the Department of Structural Engineering, Budapest University of Technology and Economics (BME). In the static load test locomotive of type M62 was applied with known geometry and axle loads with a total weight of 1120 kN, and different numbers of wagons with four axles loaded with crushed stones (Mansi et al., 2025a).

The dynamic load test was conducted to analyze the dynamic response of the bridge under the effects of moving loads with different speeds and estimate the dynamic amplification factors corresponding to different structural elements. The dynamic load test program was achieved by using two coupled M62-type locomotives moving at nearly constant speeds; a total of 15 loading cases were carried out within the speed range of 5–80 km/h, as detailed in Table 1.

The dynamic response of the bridge was estimated by measuring the strains developed in the structure by using 80 strain gauges with a sampling frequency of 75–100 Hz, located at the same position as the static load test (Mansi et al., 2025a). Two assumptions are adopted to evaluate the stress time histories measured during the test: (i) the locomotives moved at constant speed, so the distance-time relation is linear; the average vehicle speed depended on the given loading case, (ii) the environmental effects were neglected during the measurement period. Fig. 4 illustrates the dynamic responses of various structural elements at different passing speeds.

The dynamic amplification factor (DAF) was determined for each structural element separately, based on the results of both the quasi-static (5 km/h) and high-speed runs. Table 2 presents the maximum dynamic factors, which were determined from the strain gauge measurements placed on the analysed element.

3. Finite element modelling

3.1. Multi-level FE models

Effective notch stress cannot be estimated directly from the experiment, unlike the structural hot spot. Therefore, a finite element model to

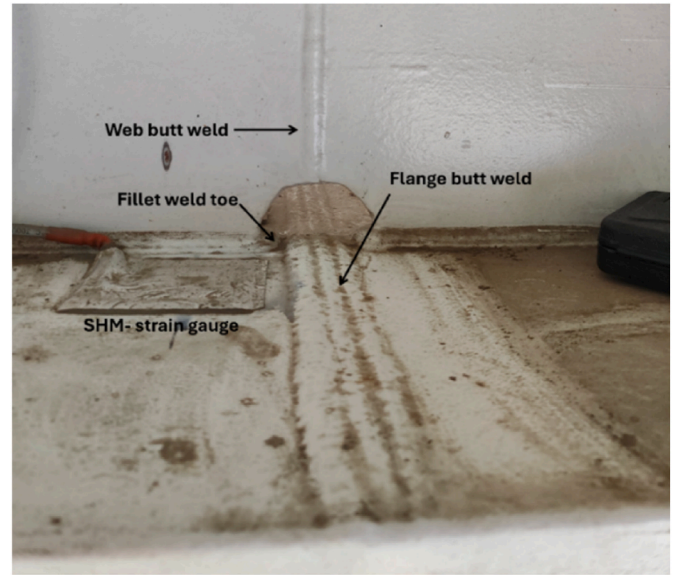


Fig. 3. Cope hole detail featuring two butt welds.

Table 1
Load cases of the dynamic test.

Load case	Speed (km/h)	Direction
1st & 2nd	5	Pest-Buda-Pest
3rd & 4th	20	Pest-Buda-Pest
5th & 6th	40	Pest-Buda-Pest
7th & 8th	60	Pest-Buda-Pest
9th & 10th	80	Pest-Buda-Pest
11th	80 km/h + braking at 85th section (between piers VI–VII)	Pest-Buda
12th	80 km/h + braking at 86th section (between piers VII–VIII)	Buda-Pest
13th	Start at full speed, accelerate to 80 km/h	Buda-Pest
14th	60 km/h + braking at the end of the bridge	Pest-Buda
15th	60 km/h + braking at 86th section (between piers VII–VIII)	Buda-Pest

evaluate the effective notch stress in the cope hole was built using ANSYS FE program (Ansys® Academic Research, 2019).

To calculate the effective notch stress of the cope hole, the computational limitations necessitated the execution of three levels of FE

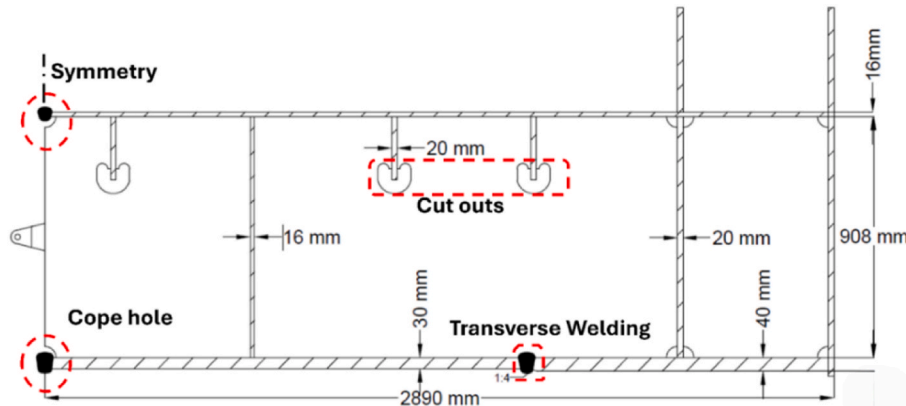


Fig. 2. Fatigue-sensitive details in the cross girder of SDRB.

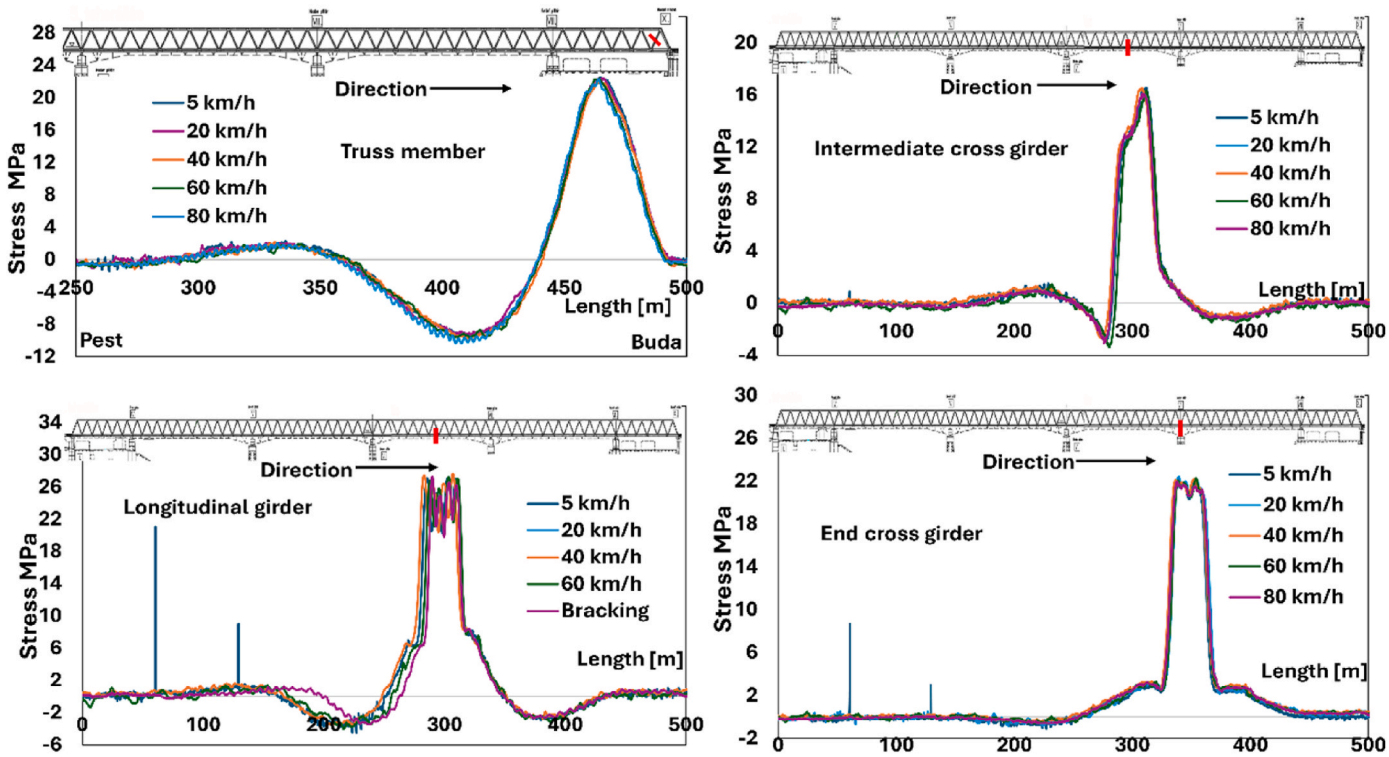


Fig. 4. Dynamic response of various structural elements.

Table 2

Maximum measured dynamic factors.

Structural element	Truss main girder	Intermediate cross girder	End cross girder	Longitudinal girder
Dynamic factor	1.028	1.048	1.045	1.049

models. A validated global shell model to simulate train movements, a local shell model of the concerned cross girder, and refined sub-models integrated within the local shell model, as shown in Fig. 5 to analyze the

stress distribution and magnitudes in the vicinity of the cope hole detail. A three-dimensional linear elastic solid element (a quadratic SOLID186 element with 20 nodes) was used to model the flange, web, and welds of the refined sub-model of the cope hole.

To achieve compatibility of displacement and stress distribution in the structural members, multi-point constraint equations are provided between different dimensions of elements. The web and flange of the refined solid model were connected to the web and flange in the shell model by using an MPC184 element, following the recommendation of coupling provided in the publications (Mashayekhi and Santini-Bell, 2019; Shim et al., 2002).

The weld geometrical parameters of the cope hole detail shown in

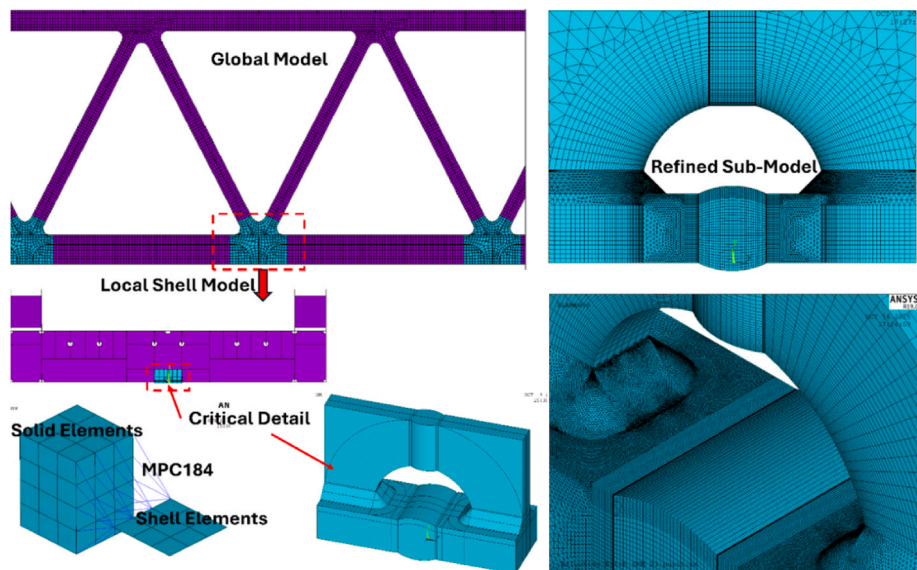


Fig. 5. Multi-level finite element models.

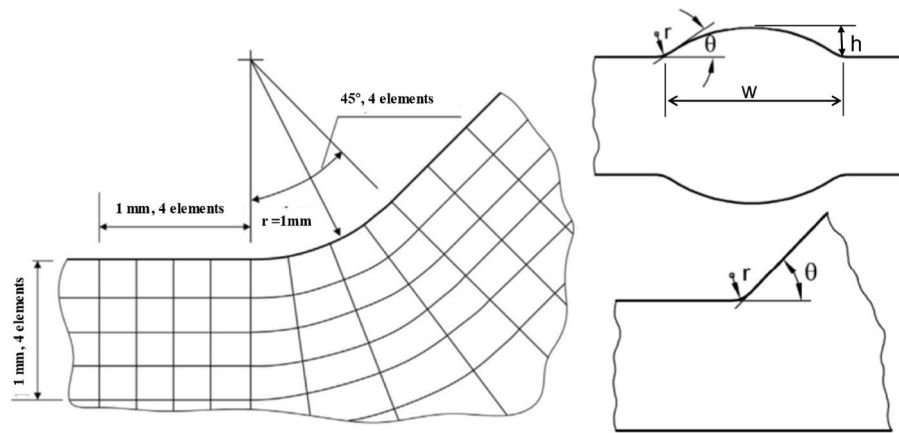


Fig. 6. Mesh details at the critical location according to IIW ([Hobbacher, 2008](#)).

Fig. 6 are measured on the actual bridge and incorporated into the numerical model. The cope hole radius is 38 mm, and the web-to-flange fillet weld size is 7 mm. For the transverse butt welds, the measured weld cap widths are 30 mm and 20 mm for the flange and web butt welds, respectively, while the corresponding weld reinforcement heights are 3.3 mm and 2.8 mm. These dimensions are used to idealize the butt weld profiles in the FE model according to the IIW effective notch stress modelling procedure. The weld toe regions are rounded using a reference fictitious notch radius of 1 mm, and the mesh geometry of weld toes is locally refined around this radius, as shown in Fig. 6, as recommended by the effective notch stress approach for welded steel details (Fricke, 2008; Hobbacher, 2008).

3.2. Model verification and validation

The verification and validation of the developed finite element model are carried out by comparing the FE results with both SHM and

static load test measurements. The SHM system provided long-term strain responses under operational train loading, while the load test offered controlled measurements under known loading conditions. The SHM stress time series recorded by strain gauges (SG), shown in Fig. 7(e) due to the passage of a 10-axle passenger train used for validation purposes. The calculated stresses at the locations of the load test sensors (Mansi et al., 2025a) are used for further validation.

The dynamic amplification factor is applied to the finite element results to amplify the static responses for validation against the SHM measurements. Fig. 7(a)–(d) illustrate a good agreement between the numerical results obtained from the FE analysis and the field measurements, confirming that the FE model accurately represents the structural behaviour in the investigated details of the bridge and can be reliably used for further analysis.

A mesh convergence study is also conducted in the refined region to determine the optimal mesh sizes necessary for precisely capturing the stress concentration at the critical location, as shown in Fig. 7(f). This

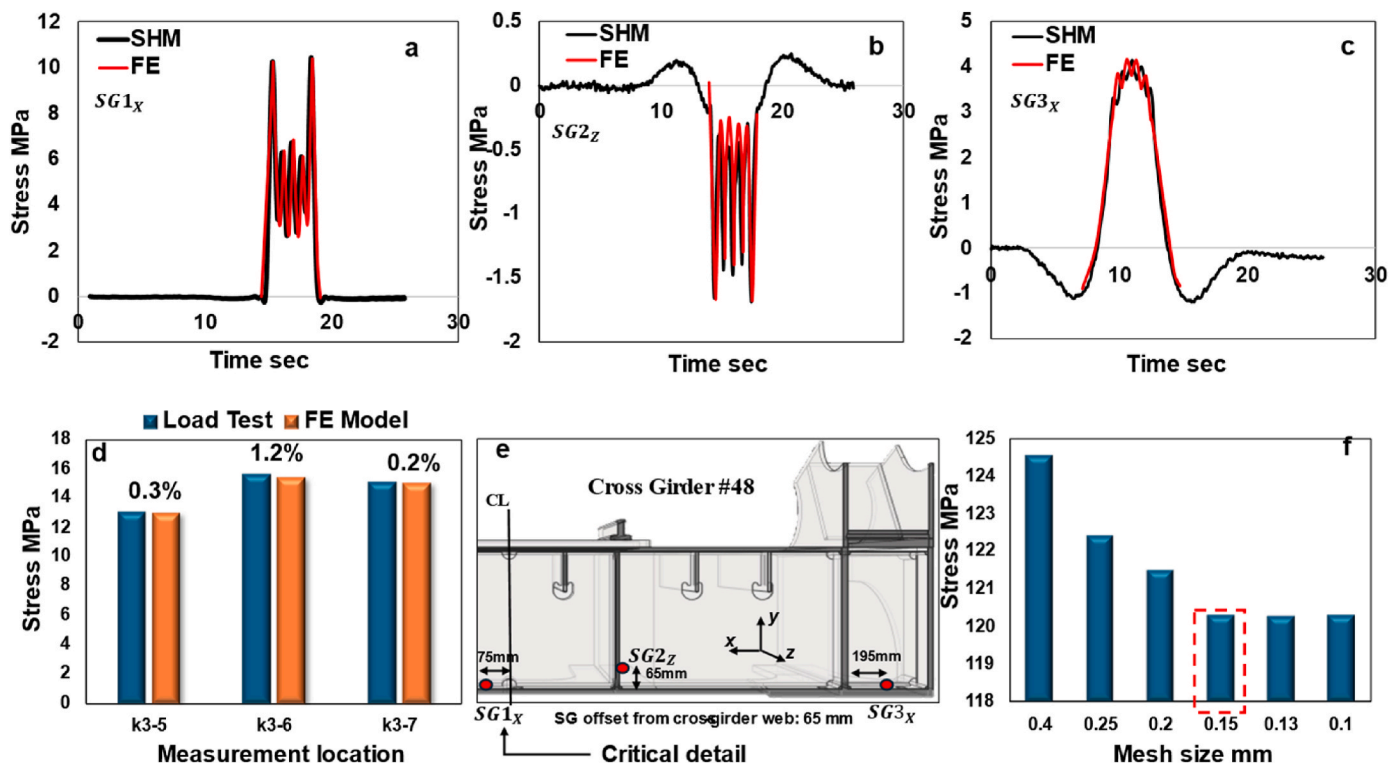


Fig. 7. Verification and validation of the FE model.

suggested that the critical region element sizes of 0.15 to 1 mm, 2 to 5 mm for the transition region, and 10 mm for the coarse region were adequate. The nodal displacements at the connection points of the global model with the local shell model are imposed as a boundary condition on the local model to mitigate the impact on the effective notch stress concentration.

4. Calculation of stress concentration factor

To calculate fatigue damage at the sensitive detail, the nominal stress and effective notch stress methods are adopted. The SHM strain gauge is installed near the cope hole and continuously measures the stress responses under operational loading conditions. The cope hole geometry features two butt welds: the first is a transverse weld on the bottom flange, while the second is a transverse weld on the web of the cross girder, in addition to the fillet weld between the web and the flange, as shown in Fig. 3.

Finite element analysis is employed to calculate stress concentrations induced by trains. Four typical trains with varying loads and geometric configurations are considered for analysis, as shown in Fig. 8. The study conducted more than 150 FE analyses under different load conditions to evaluate the stress concentration locations. Fig. 9 illustrates the hot spot points produced within the investigated detail under various load conditions. Three hot spot points are identified in the local detail: HS1 at the fillet weld toe, HS2 at the butt weld toe on the web side, and HS3 at the butt weld toe on the flange side. The predicted location of crack initiation is at the intersection point of the cope hole and fillet weld HS1 (British Standards Institution, 2025; Hobbacher, 2008; Wei et al., 2017), but the presence of the butt weld on the web side of the cope hole may turn the crack initiation point to HS2 (Cai et al., 2017). The stress concentrations at the hot spot HS3 are much less than those at HS1 and HS2. The maximum principal stress values relative to the stress of HS2 are 0.94, 1.0, and 0.56 for HS1, HS2, and HS3, respectively, and that is

consistent with the results of previous studies (Cai et al., 2017; Miki and Tateishi, 1997; Xiao and Yamada, 2005). Consequently, the HS3 is considered not a serious critical point and excluded from further analysis.

The stress concentration factor is defined as the ratio between a specified FE-calculated stress and the measured stress at the strain-gauge location. The specified stress may represent the nominal stress in the flange or the local critical stress at HS1 or HS2. Accordingly, the numerically determined factors are used to convert the stress time history measured by the strain gauge into the corresponding nominal stress time history in the flange and the local stress time histories at the critical hot-spot locations. Fig. 10 illustrates the regression analysis performed between the stress values calculated at the strain gauge location and the nominal stresses at the bottom flange, as well as the maximum principal and von Mises stresses experienced at the hot spots (HS1 and HS2). The strong linear relationship with a high coefficient of determination ($R^2 = 0.988$) provided the best representation of the plotted data, confirming the reliability and consistency of the detected trend. The analysis yields a SCF of 1.11 for the nominal stress. In contrast, the hot spot points reveal von Mises SCFs of 4.19 and 4.35, and maximum principal SCFs of 4.7 and 4.99 for HS1 and HS2, respectively. The results indicate that HS2 experiences higher stress concentration than HS1, mainly due to the presence of the web-side butt weld in the cope hole region. Therefore, although HS1 corresponds to the conventional fatigue-critical location assumed in standard cope hole classifications, HS2 may govern the fatigue assessment of the investigated non-standard detail.

5. Fatigue life assessment

5.1. Nominal stress method

The nominal stress method evaluates the fatigue performance of the

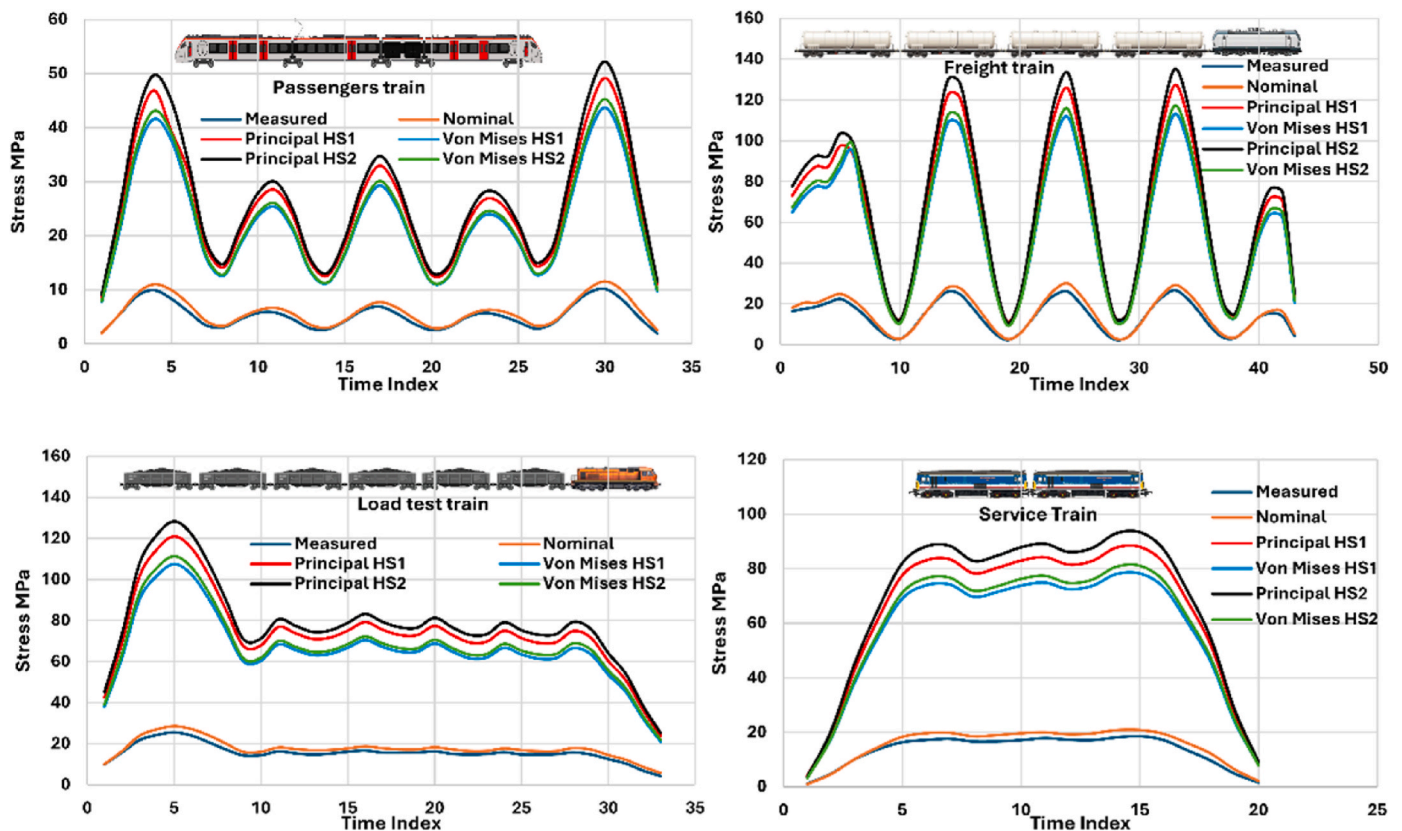


Fig. 8. Measured, nominal, von Mises, and principal stress responses under passing trains.

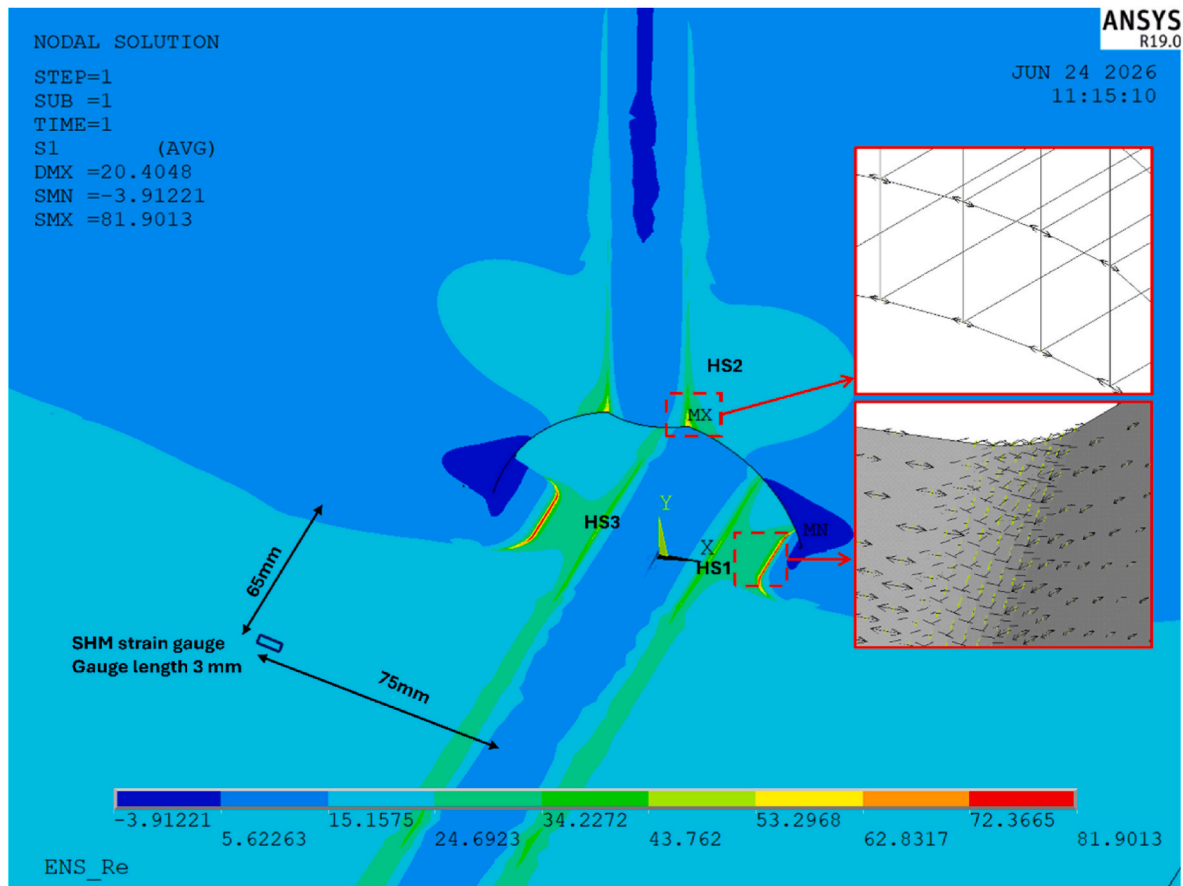


Fig. 9. HS points and maximum principal stress distribution.

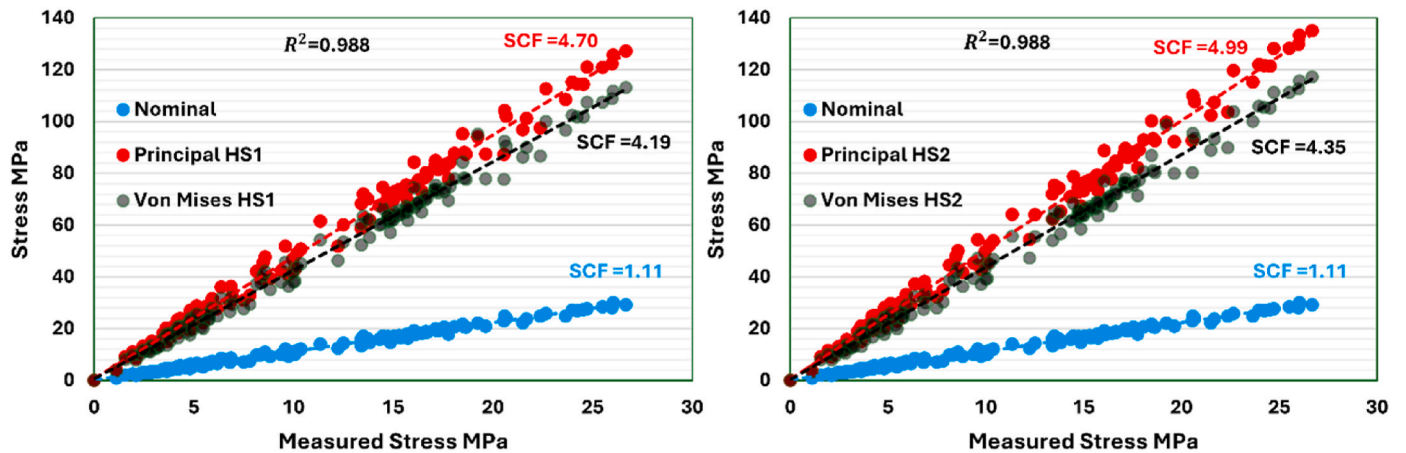


Fig. 10. Regression analysis for the calculation of SCF.

structural details using the nominal stress calculated at the critical sections. The cope hole welded joints with a transverse butt weld in the flange are classified as FAT 71 category according to EN 1993-1-9 and the IIW recommendations (British Standards Institution, 2025; Hobbacher, 2008). Although the cope hole welded joints feature a butt weld configuration on the flange and web sides, which are not addressed in current design standards, the state of practice in Hungary does utilize this type of connection in most recently constructed railway steel bridges. An experimental study investigated a cope hole detail consisting of a butt weld on the web side only, incorporated into a welded truss bridge in China, indicating that FAT 63 is the most closely related

fatigue class curve for these types of welded joints (Cai et al., 2017). Therefore, in the current study, the nominal stresses developed in a cope hole featuring butt welds on the flange and web sides were first estimated and represented by stress-range spectra of the actual train-induced loading. The nominal stress is calculated along section (A) using the FE model, as shown in Fig. 11. The SHM strain gauge continuously measures the train-induced stress time history, which then converts to a nominal stress time history using the SCF, which is calculated in the previous section.

The measured and the calculated nominal stress ranges (by averaging the stresses within the flange in the studied cross-section) and

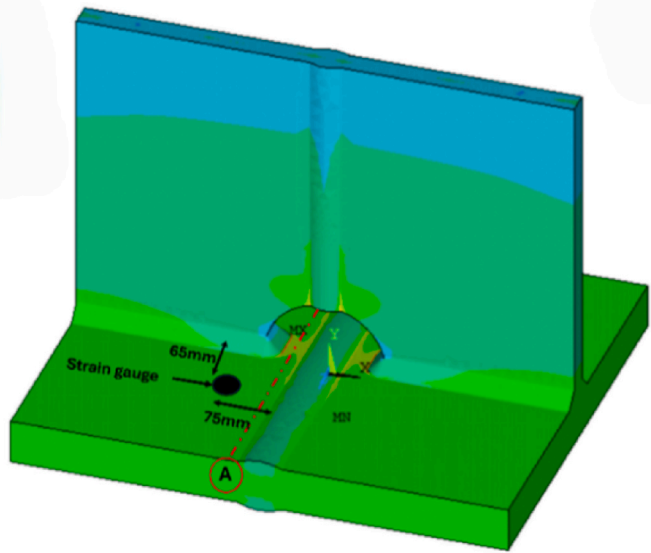


Fig. 11. Section of estimation of the nominal stress.

their cycles under real traffic conditions are estimated using the rainflow counting method, as described in (Downing and Socie, 1982). Fig. 12 illustrates the measured and nominal stress-time histories experienced in the investigated detail, along with their relevant stress ranges and cycles histograms, as monitored over a typical day.

The fatigue performance of the concerned cope hole was evaluated by comparing the nominal stress ranges within the appropriate FAT class provided in the relevant design codes FAT 71 class according to (British Standards Institution, 2025; Hobbacher, 2008). On the other hand, the IIW recommendations consider the interaction between the shear stress in the web and the normal stress in the flange at the weld end. Accordingly, the fatigue resistance of cope-hole details decreases as the

shear-to-normal stress ratio increases. Since the SDRB consists of three separated superstructures arranged side by side, with each superstructure carrying one railway track and the FE analysis was performed within the linear-elastic range, changes in load magnitude produce proportional changes in both the web shear stress and the flange normal stress. Therefore, the shear-to-normal stress ratio was evaluated using FE results from the considered loading cases, including different representative train configurations with varying axle loads, axle spacings, and longitudinal load positions along the track. Accordingly, the relationship shown in Fig. 13 corresponds to the obtained stress-range ratio, $\Delta\tau/\Delta\sigma = 0.11$, indicating that the applicable fatigue class should be reduced to FAT 63 according to the IIW recommendations. Therefore, the fatigue assessment of the cope-hole detail was also performed using

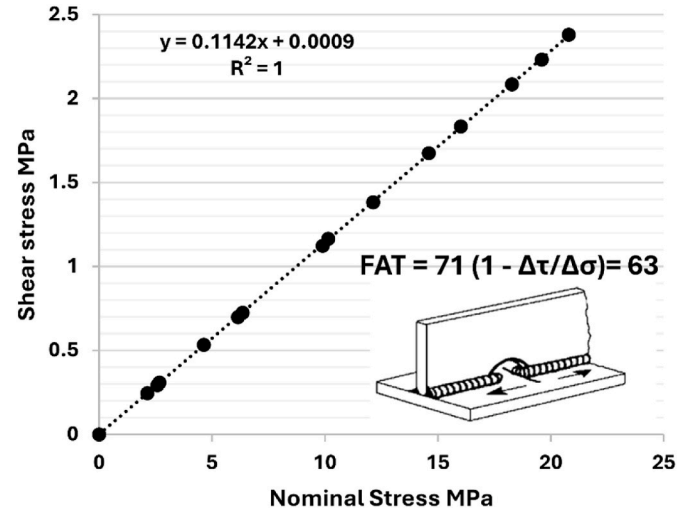


Fig. 13. Shear to nominal stress ratio at the cope hole detail.

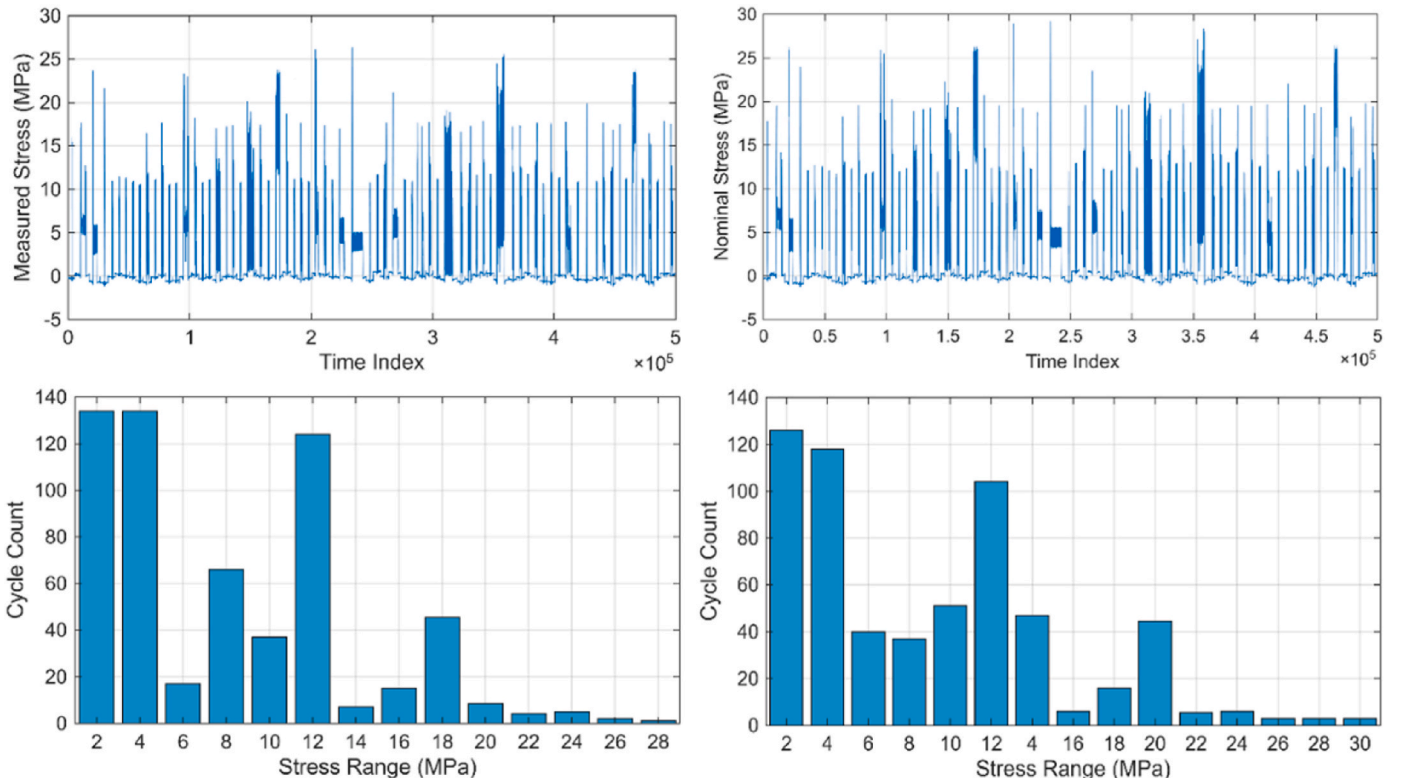


Fig. 12. Measured and nominal stress-time histories and the corresponding cycles.

FAT 63.

Fig. 14 illustrates two key considerations through fatigue damage calculations. The Eurocode S-N curve (blue line) consists of two slopes, $m = 3$ and $m = 5$, with the knee point stress corresponding to 5 million cycles; and the stress ranges below the cut-off limit do not contribute to fatigue damage. While the IIW S-N curve (red dashed line), is represented by two slopes, $m = 3$ and $m = 22$, with the knee point stress corresponding to 10 million cycles. Therefore, fatigue damage is evaluated using the FAT 71 and FAT 63 characteristic design fatigue resistance curves. Therefore, they provide a conservative fatigue assessment of the investigated detail. For comparison, the corresponding mean FAT values may be estimated by multiplying the characteristic FAT values by 1.37 (Neuhäusler et al., 2025). Accordingly, FAT 71 and FAT 63 correspond approximately to mean FAT values of 97 MPa and 86 MPa, respectively. Since the fatigue damage is highly sensitive to the selected FAT value, the use of mean FAT values would reduce the calculated accumulated damage D and increase the estimated fatigue life. Nevertheless, the characteristic FAT classes were retained as the main basis of the assessment because they are consistent with the Eurocode design verification approach and provide a conservative evaluation of the fatigue safety of the bridge detail. Miner's linear damage rule is used to accumulate the damage, where each stress range contributes to the total damage according to the relevant S-N curve of the selected FAT category, using the following equations:

$$D_1 = \sum D_i = \sum n_i \frac{(\Delta\sigma_{R,i})^m}{N_D \times \Delta\sigma_D^m} \text{ for } \Delta\sigma_R < \Delta\sigma_D \quad (1)$$

$$D_2 = \sum D_i = \sum n_i \frac{(\Delta\sigma_{R,i})^m}{2 \times 10^6 \times \Delta\sigma_C^m} \text{ for } \Delta\sigma_R \geq \Delta\sigma_D \quad (2)$$

$$\text{Total damage } D = D_1 + D_2 \quad (3)$$

where:

$\Delta\sigma_{R,i}$ and n_i are the applied stress range and the corresponding number of cycles,

$\Delta\sigma_D$ and N_D are the knee point stress and the corresponding number of cycles.

5.2. Effective notch stress method

The fatigue assessment of welded joints based on effective notch stress utilizes two universal S-N curves: FAT 225, evaluated based on the maximum principal stress, and FAT 200, evaluated based on the von Mises stress (Fass et al., 2023; Hobbacher, 2008). Therefore, the measured stress-time histories obtained from the strain gauge are

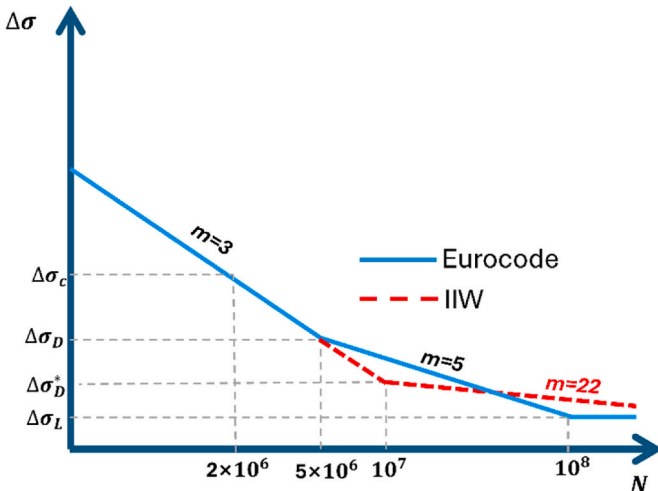


Fig. 14. Interpretation of nominal stress S-N curves.

converted to the maximum principal stress and von Mises time histories at the hot spot points, using the SCFs estimated in Section 4, as detailed in Eq. (4).

$$\sigma_{HS}(t) = SCF \sigma_{st}(t) \quad (4)$$

where:

$\sigma_p(t)$ effective stress time history at the concern hot spot point,

$\sigma_{st}(t)$ stress time history measured by the strain gauge.

The maximum principal and von Mises stress ranges, and their cycles under real traffic conditions, are evaluated using the rainflow counting method (Downing and Socie, 1982). Fig. 15 illustrates the maximum principal stress and von Mises stress time histories at the hot spot point (HS2), along with their relevant stress ranges and cycles histograms, as calculated over a typical day. The cycles corresponding to stress ranges smaller than 2 MPa are ignored. It is evident that the spectrum encompasses a wide range of stresses, spanning from 2 MPa to a maximum value of approximately 140 MPa. Although the SCF values are close, the slightly higher stress range at HS2 is important because fatigue damage is highly sensitive to stress range. Therefore, even a small increase in stress range can lead to a noticeable increase in accumulated fatigue damage. The results indicate that HS2 governs the fatigue performance of the investigated cope hole detail. This finding suggests that the standard assumption that HS1, located at the conventional cope hole weld toe region, is always the fatigue-critical point may not be valid for this non-standard weld configuration. In the present detail, the additional web-side butt weld modifies the local stress distribution and shifts the most critical fatigue location from HS1 to HS2. Therefore, the relatively higher SCF and stress ranges at HS2 were adopted for the fatigue damage calculations.

Fatigue classes FAT 225 and FAT 200 S-N curves, recommended by (British Standards Institution, 2025; Fricke, 2008), are employed for the fatigue assessment based on maximum principal stress and von Mises stress, respectively. Fatigue damage is calculated based on the two-slope relationships indicated by the curve. The curve consists of high- and low-cycle portions, which are used for damage assessment of stress ranges below and above the knee point corresponding to 10 million cycles ($\Delta\sigma_D = 130.57, 116.96$ MPa) for FAT225 and FAT200, respectively, as shown in Fig. 16. The fatigue damage D_1 induced by stress ranges lower than the knee point is calculated using Eq. (5), while the fatigue damage D_2 caused by stress ranges greater than knee point is estimated using Eq. (6). The total fatigue damage represents the sum of D_1 and D_2 .

$$D_1 = \sum D_i = \sum n_i \frac{(\Delta\sigma_{en,i})^{22}}{10^7 \times \Delta\sigma_D^{22}} \quad (5)$$

$$D_2 = \sum D_i = \sum n_i \frac{(\Delta\sigma_{en,i})^3}{2 \times 10^6 \times \Delta\sigma_C^3} \quad (6)$$

where:

$\Delta\sigma_{en,i}$ effective notch stress (principal or von Mises stress),

$\Delta\sigma_D$ stress range corresponding to the knee point,

$\Delta\sigma_C$ reference stress range corresponding to 2.0 million cycles,

n_i number of cycles corresponding to the stress range $\Delta\sigma_{en,i}$,

D_i fatigue damage.

5.3. Fatigue damage assessment

Fatigue damage assessment of the cope hole detail was performed using the SHM strain-gauge data recorded between December 2021 and December 2024. The measured strain histories were transformed into nominal and local effective notch stress histories, allowing the fatigue response of the investigated detail to be evaluated under real operational railway traffic. This highlights the importance of the SHM measurements, since they provide direct information on the actual stress

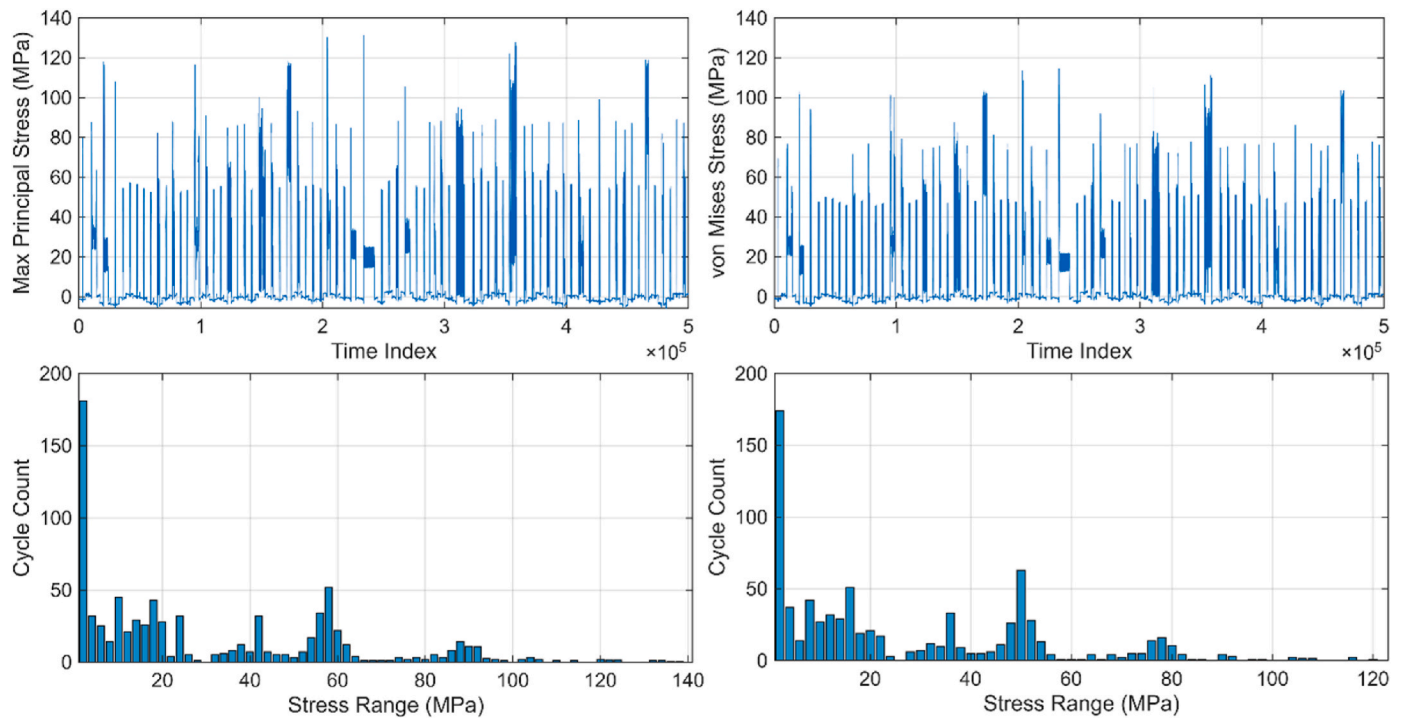


Fig. 15. Principal and von Mises stress time histories and corresponding cycles at detail HS2.

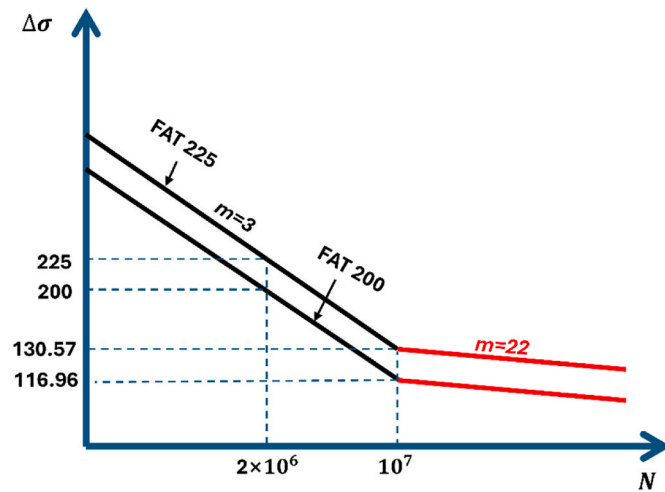


Fig. 16. S-N curves of FAT225 and FAT200 classes according to IIW (Fricke, 2008).

ranges experienced by the bridge rather than relying only on assumed design traffic or simplified load models. The distribution of the nominal stress ranges shows that most cycles are below the knee point of the S-N curve and close to the cut-off limit. This indicates that the bridge detail is mainly subjected to low stress-range cycles. Consequently, the fatigue demand under normal service loading is relatively low, suggesting that the bridge possesses sufficient fatigue resistance.

Table 3 presents a comparative evaluation of the daily fatigue damage caused by operational loads using the different fatigue classes and S-N curves discussed in the previous sections. The results show a pronounced variation among the different assessment approaches, demonstrating the strong sensitivity of the calculated damage to the selected S-N curve, fatigue class, and stress evaluation method. The nominal stress assessment based on the IIW S-N curve gives lower damage, even when the fatigue class is reduced to FAT 63. This is mainly

Table 3

Daily fatigue damage based on different methods and S-N curves.

Fatigue class and S-N curve	Daily damage $\times 10^{-8}$
FAT 71 IIW	0.097
FAT 71 Eurocode	5.9
FAT 63 IIW	1.35
FAT 63 Eurocode	58.3
FAT 200 IIW	69.0
FAT 225 IIW	93.0

because several stress ranges that contribute to damage along the $m = 5$ branch of the Eurocode S-N curve are shifted to the very-high-cycle region with $m = 22$ in the IIW S-N curve. As a result, these cycles contribute much less to the accumulated damage, leading to a lower predicted fatigue damage. In contrast, the effective notch stress assessment produces higher damage values because it directly captures the local stress concentration at the critical notch locations. The damage was evaluated using the maximum principal stress with FAT 225 and the equivalent von Mises stress with FAT 200. The maximum principal stress-based assessment gives the highest fatigue damage and therefore indicates the shortest fatigue life. This is attributed to the higher local stress concentration and the larger stress-range cycles obtained at the critical hot-spot location. Since fatigue damage is highly sensitive to stress range, even a moderate increase in the local stress range can significantly increase the accumulated damage. The comparison also shows that the Eurocode FAT 63 nominal stress curve gives the closest damage estimate to the effective notch stress-based assessments. This indicates that reducing the nominal fatigue class can partly account for the increased local fatigue demand at the investigated cope hole detail. However, the effective notch stress results provide a more direct interpretation of the local fatigue behaviour, especially for the non-standard weld configuration considered in this study. The results show that the predicted fatigue performance is strongly influenced by the selected S-N curve and the stress evaluation method. For complex weld details (such as the subject of the current study), the governing fatigue-critical location should be verified rather than assumed based solely on standard

detail classifications.

Fig. 17 illustrates the maximum principal daily stress range amplitude spectrum estimated by the rainflow cycle counting method, along with the corresponding fatigue damage at HS2 calculated using Eqs. (5) and (6). The distribution indicates that the number of higher-amplitude cycles is limited, while the low-amplitude cycles represent the majority of the stress cycle spectrum, causing less fatigue damage. Nevertheless, the majority of fatigue damage is produced by medium-amplitude cycles, which is consistent with the overall trend of fatigue damage in welded details.

The maximum principal stress-based fatigue damage calculation is extended to consider the entire monitoring period. The linear accumulation method is used to estimate fatigue damage on a daily basis. Fig. 18 illustrates the daily fatigue damage corresponding to the monitoring period. It is noticeable that the more serious damage occurred during the interval from June 2022 to February 2023, due to the higher train loads experienced on the bridge. Therefore, some data gaps appear in the results, which occurred because no measurements were recorded during those periods. Since this superstructure represents one of three parallel structures, while the other two remained in operation, the missing data likely correspond to periods when this bridge was temporarily closed for maintenance or operational reasons.

Daily fatigue damage was accumulated over the monitoring period to calculate the total fatigue damage experienced at the critical detail. Fig. 18 reveals a non-proportional trend of damage accumulation, which is attributed to the intrinsic variability of operational railway traffic, including variations in train frequency and load intensity. The slope of the accumulated fatigue damage curve increases during periods of higher traffic demand, while it decreases during periods of reduced traffic. Several discontinuities are also observed in the cumulative damage curve where no stress cycles were recorded. These results show that cumulative fatigue damage under real traffic does not develop uniformly with time, but depends strongly on the actual operational loading history. Therefore, long-term SHM measurements are essential for capturing the real stress cycles experienced by fatigue-sensitive details and for evaluating their damage accumulation under actual service

conditions. Moreover, the proposed methodology can be extended to several critical bridge details simultaneously by combining measured strain responses with calibrated FE-based stress transformation factors. This allows the stress cycles and accumulated fatigue damage at different details to be assessed and compared. This enables engineers to identify which details experience higher stress ranges or faster damage accumulation and, consequently, to prioritize these locations for inspection, continued monitoring, or preventive maintenance once a specified number of stress cycles or accumulated fatigue damage level is reached.

6. Conclusions

This study investigated a comprehensive SHM-based fatigue assessment approach for a non-standard cope hole detail in the Southern Danube Railway Bridge (SDRB) in Budapest. The investigated detail contains transverse butt welds in both the flange and web plates, which makes its local fatigue behaviour different from conventional cope hole configurations. The study integrated refined finite element modelling with long-term SHM strain-gauge measurements recorded under actual railway traffic. The main conclusions are summarized as follows:

- Three critical locations were investigated: HS1 at the fillet weld toe near the cope hole region, HS2 at the butt weld toe on the web side, and HS3 at the butt weld toe on the flange side. HS1 and HS2 were found to be the most structurally significant locations, while HS3 showed lower fatigue sensitivity.
- The stress concentration factors between the measured stress and the evaluated nominal, maximum principal, and von Mises notch stresses were determined. The nominal stress concentration factor in the flange was 1.11, while the von Mises stress concentration factors were 4.19 and 4.35 for HS1 and HS2, respectively. The corresponding maximum principal stress concentration factors were 4.70 and 4.99 for HS1 and HS2, respectively.
- Although the effective notch stress results at HS1 and HS2 were relatively close, HS2 showed slightly higher stress concentration and

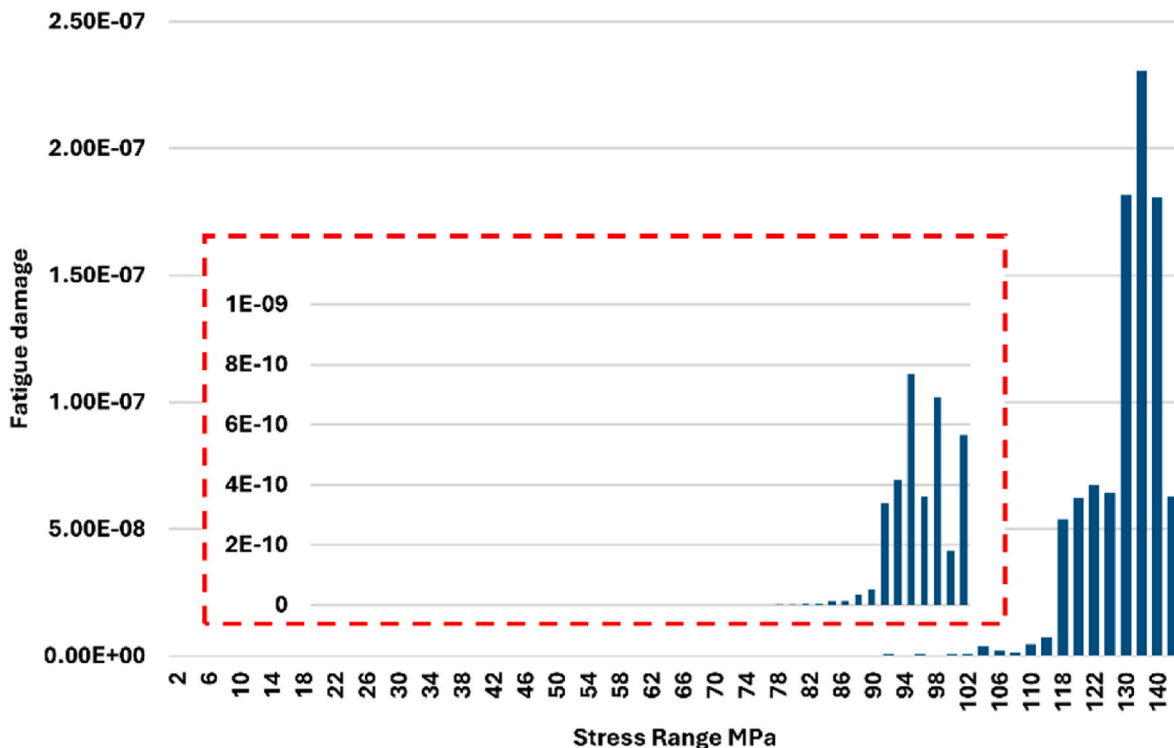


Fig. 17. Daily stress ranges and the corresponding damage.

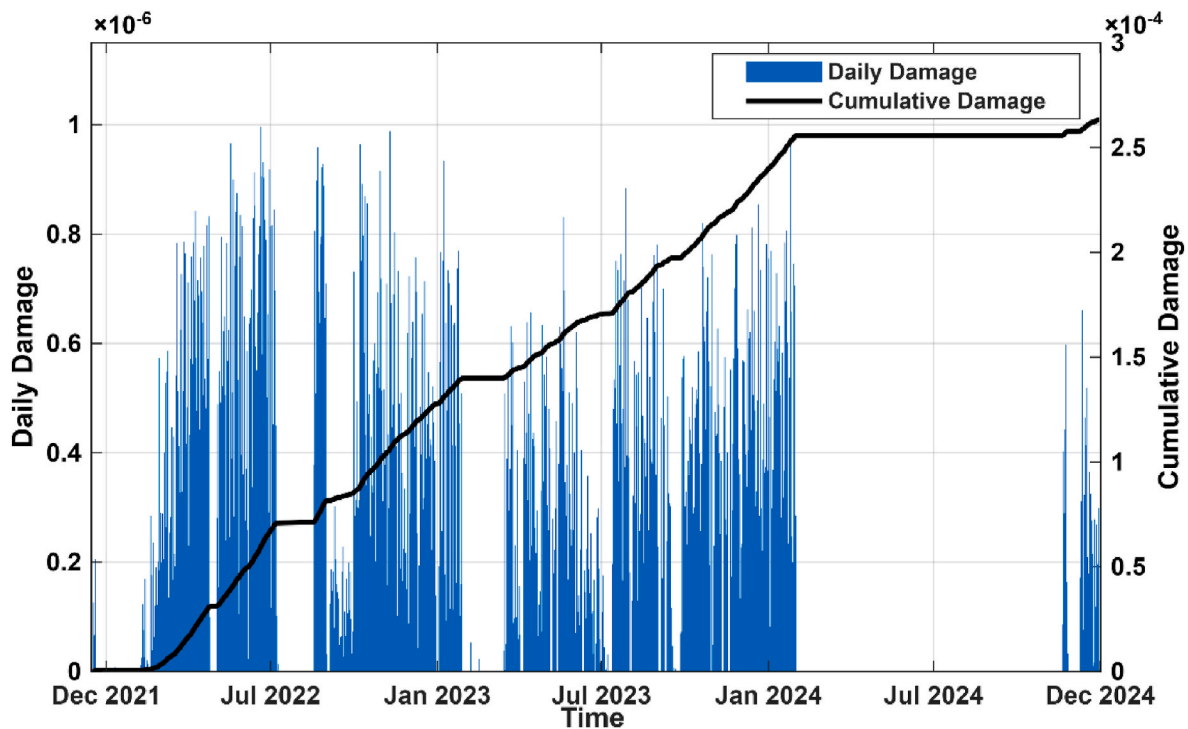


Fig. 18. Daily and cumulative damage at HS2 over the monitoring period.

stress-range cycles. Since fatigue damage is highly sensitive to stress range, this resulted in higher fatigue damage at HS2. This finding indicates that the conventional assumption that HS1 is always the fatigue-critical location in cope hole details is not necessarily valid for non-standard weld configurations. For the investigated detail, HS2 governs the fatigue performance.

- The fatigue damage assessment showed that the predicted damage is highly sensitive to the selected S–N curve and stress evaluation method. The nominal stress-based assessment using FAT 71 and FAT 63 according to Eurocode and IIW produced different damage levels due to the different slopes, knee points, and cut-off assumptions of the corresponding S–N curves.
- The effective notch stress-based assessment using FAT 225 for the maximum principal stress and FAT 200 for the equivalent von Mises stress produced higher fatigue damage than the nominal stress-based assessment. The maximum principal stress-based evaluation resulted in the most severe daily damage due to the higher local stress concentration and larger stress-range cycles at the critical location.
- Overall, the findings emphasize that the fatigue assessment of non-standard welded bridge details should not rely only on conventional fatigue classifications or predefined critical locations. The selected S–N curve, stress evaluation method, local weld configuration, and actual measured stress-range distribution should be carefully considered, since these factors can significantly influence the predicted fatigue damage and may change the governing fatigue-critical location. Additionally, the proposed methodology is not limited to the investigated cope hole detail. It can be extended to several fatigue-sensitive bridge details simultaneously by combining measured strain responses with calibrated FE-based stress transformation factors. This enables engineers to identify details subjected to higher stress ranges or faster damage accumulation and to prioritize them for inspection, continued monitoring, or preventive maintenance once a specified number of stress cycles or accumulated fatigue damage level is reached.

CRediT authorship contribution statement

Ali Mansi: Writing – original draft, Methodology, Investigation, Formal analysis, Data curation. **Balázs Kövesdi:** Writing – review & editing, Validation, Conceptualization. **László Dunai:** Writing – review & editing, Visualization, Supervision, Resources.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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