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A REVIEW OF DAMAGE TYPES, STRESS CONCENTRATION AND FATIGUE EFFECTS IN EVALUATING THE RESIDUAL LOAD-CARRYING CAPACITY OF DEFECTIVE STEEL GIRDERS IN RAILWAY BRIDGES

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Abstract. Steel girders of ageing railway bridges deteriorate through corrosion, perforation, loose fasteners and local deformation; in Vietnam, 465 of 1,862 railway bridges are classified as vulnerable and their residual capacity must be re-evaluated. Inspection practice rates such defects by visual condition and area loss, whereas most act through local stress concentration, which governs fatigue life and is invisible to global measurements; this gap has not been quantified. This paper reviews the common damage types and quantifies their effect through two mechanisms, area loss and stress concentration, using classical elasticity solutions, the fatigue notch factor and the power-law S-N relation; examines TCVN 14478:2025, Eurocode 3, AASHTO LRFD and AISC 360; and re-analyses the inspection data of a 40-year-old steel truss railway bridge under local stress conditions. For perforations and loose fasteners the stress concentration factor reaches 2.5 to 3.5, two to three times the effect of area loss, and reduces fatigue life to a few percent of the intact value; none of the four standards quantifies it. On the bridge, the nominal stress at a member with a missing bolt is 57 % of the design strength and the frequency change lies within measurement error, yet the local stress at the hole edge exceeds the yield strength and the fatigue life there falls by one to two orders of magnitude. A look-up table of stress concentration factors, a screening factor, a fatigue check with the notch factor and recommended actions are proposed as a screening basis for inspection, pending validation.

Keywords: steel girder, railway bridge, defect, stress concentration, fatigue, inspection standard.

1. INTRODUCTION

Steel girders are critical load-carrying members in railway bridge systems. A large proportion of steel bridges in service are of advanced age and require a comprehensive re-evaluation of their load-carrying capacity. In the United States, the Federal Railroad Administration requires track owners to implement bridge management programmes that include annual inspections of railroad bridges and to know the safe load capacity of each bridge [1]. In Europe, most riveted steel bridges have exceeded 100 years of age and are in a stage requiring re-evaluation [2].

In Vietnam, the deterioration of the railway steel bridge system is a pressing issue. According to a 2024 report of the Vietnam Railway Corporation, announced through the press [3-5], there are 465 vulnerable bridges out of a total of 1,862 bridges across the network; many bridges have exceeded their service life or have heavily corroded steel members and weak decks, with the cost of repairing the vulnerable bridges estimated at about 4,028 billion VND [3,4]. Within the demand for emergency reinforcement of works with very high risk of unsafety, the portion for 94 bridges alone is about 700 billion VND [3,5], carried out by measures such as adding splice plates, replacing structural members with severe perforation corrosion, replacing rivets with decayed heads, and replacing damaged bolts [3]. In addition to time-dependent deterioration, old steel bridges also face the risk of sudden damage due to collision, as exemplified by the 2024 incident at Tam Bac bridge in Hai Phong, a steel bridge built in 1902 and shared by railway and road traffic, which was struck by a barge causing structural deformation and disruption of operation [6]. Fig. 1 illustrates two typical damage types of steel girders in railway bridges.

Figure 1. Typical damage of steel girders in railway bridges: (a) web perforation and rivet-hole perforation at the flange (source: Bao Xay Dung [3]); (b) corrosion of the top flange causing area loss at Tam Bac bridge, Hai Phong (source: Bao Cong Ly [6]). The press sources [3,6] document the occurrence of the damage only, not technical data.

Accurate assessment of the residual load-carrying capacity of damaged steel girders is essential for bridge management and maintenance. The issue is that current assessment methods are mainly based on cross-sectional area loss and nominal stress, whereas many common defects pose a hazard primarily through the local stress concentration effect, which strongly affects both static strength and fatigue life but is difficult to detect by global measurement methods.

On this basis, the paper is carried out with four objectives. First, to classify the common damage types of steel girders in railway bridges and to quantitatively analyse their effects on load-carrying capacity and fatigue life according to two mechanisms, area loss and stress concentration. Second, to analyse the gaps in the Vietnamese inspection standard TCVN

14478:2025 and in international design standards regarding the quantitative consideration of local defects. Third, through an analysis of inspection data from an actual steel truss railway bridge, recomputing the measured values under local stress conditions to show that the current assessment method does not fully reflect the working state at defect locations and tends to overestimate the load-carrying capacity. Fourth, to propose supplementary assessment criteria based on the quantitative analyses presented.

2. THEORETICAL BASIS OF THE EFFECT OF DEFECTS

2.1. Two degradation mechanisms and the stress concentration factor

The residual load-carrying capacity of a defective steel member is governed by two mechanisms that are different in nature. The first mechanism is cross-sectional area loss, which increases the nominal stress in inverse proportion to the remaining area. If the intact area is A_0 and the lost area is ΔA, the nominal stress increases by a factor $A_0/(A_0 - \Delta A)$. For most local defects, the area loss is small; for instance, a circular hole of 40 mm diameter on a flange of 200 mm width reduces only 20 % of the width at one cross-section.

The second mechanism is local stress concentration. The stress concentration factor is a dimensionless quantity characterising the magnification of stress in the vicinity of a geometric discontinuity such as a hole, notch or abrupt change of section, defined as the ratio between the maximum local stress and the nominal stress at the considered section [7]:

$$K_t = \frac{\sigma_{max}}{\sigma_{nom}} \tag{1}$$

where σ_{max} is the maximum stress at the edge of the discontinuity and σ_{nom} is the nominal stress, usually computed on the net section. The factor K_t depends only on the shape of the discontinuity and the loading condition, not on the absolute size, and is a characteristic of the elastic stress field [7].

A comparison of the two mechanisms reveals the core of the problem. For the same circular hole, the local area loss increases the nominal stress by a factor of about 1.25 (corresponding to a 20 % width loss), whereas stress concentration increases the local stress by a factor K_t reaching 3.0. Thus, for a hole-type defect, the stress concentration effect is about two to three times greater than the area-loss effect, and this is the factor commonly neglected in area-based assessment methods.

2.2. Stress concentration at perforations and corrosion

Surface corrosion is the most frequently recorded damage type in steel bridge structures, caused by electrochemical reactions between the steel surface and the moist environment. According to Albrecht and Naeemi [8], corrosion can cause a loss of 3 to 10 % of the cross-sectional area after about 20 to 30 years of service. A distinction must be made between uniform corrosion, distributed relatively evenly and causing negligible stress concentration with K_t from 1.0 to 1.2, and localised pitting corrosion, forming local pits with K_t from about 1.2 for pits with an elongated mouth to about 3.0 for the sharpest pits with a circular mouth, and about 2.25 for a hemispherical pit, while the ratio of pit depth to plate thickness has a negligible effect as long as it does not exceed 0.2 [9]. Fernandez et al. [10] showed that pitting corrosion strongly affects fatigue life by creating crack-initiation points, while Nishikata et al. [11] emphasised the severity of corrosion in coastal areas. In Vietnam, Mac and Nguyen [12] surveyed postcorrosion strength, fatigue and stability at the connections of the Cho Thuong weathering steel railway bridge, confirming that corrosion agents accumulate most strongly at connections.

When corrosion progresses through the entire plate thickness, a perforation is formed. The stress concentration effect at a hole has been studied in classical elasticity theory. For a circular hole in an infinite flat plate under uniform tension, the Kirsch solution [13] gives:

$$K_t = 3.0 \tag{2}$$

For a plate of finite width W with a circular hole of diameter d, the stress concentration factor referred to the nominal stress on the net section is expressed by the empirical formula [7]:

$$K_t = 3.00 - 3.14\left(\frac{d}{W}\right) + 3.667\left(\frac{d}{W}\right)^2 - 1.527\left(\frac{d}{W}\right)^3 \tag{3}$$

For an elliptical hole with semi-axis a perpendicular to the tensile stress direction and tip radius of curvature ρ, the Inglis solution [14] gives:

$$K_t = 1 + 2\sqrt{\frac{a}{\rho}} \tag{4}$$

For a circular hole, when $a = \rho$ then $K_t = 3.0$, consistent with the Kirsch solution. As the hole flattens along the stress direction, the radius of curvature ρ decreases and the stress concentration factor increases rapidly. On an experimental basis, Bao et al. [15] studied the residual web bearing capacity of corroded steel girders, modelling two forms, web thinning and web perforation; the results showed that the residual bearing capacity can drop to only 30 to 50 % of the original value in severe corrosion cases, confirming that the hazard of a perforation far exceeds the area it occupies.

2.3. Stress concentration at loose bolt and rivet connections

Rivet relaxation and bolt loosening are damage types characteristic of aged steel bridges, formed by corrosion reducing the connector shank cross-section, loss of pretension under long-term cyclic loading, and loosening due to continuous vibration [16]. When the connection is still tight, the clamping force between the steel plates transmits the load through friction and distributes the stress relatively evenly around the hole. When the connection loses its clamping force, the load is transmitted through bearing contact at the hole wall, bringing the hole close to the free-hole state and causing severe stress concentration. This is a particularly hazardous damage type because it causes almost no cross-sectional area loss yet produces a high stress concentration factor.

Imam et al. [2,17] comprehensively studied the fatigue behaviour of riveted railway bridges in the United Kingdom using a global-local analysis combined with the Theory of Critical Distances. The results showed that the stress concentration factor at the connection increases from about 1.5 to 2.0 when the rivet is still tight to 3.0 to 3.5 when the rivet is fully loose. Bruhwiler and Kunz [18], through fatigue tests on girders dismantled from a 91-year-old railway bridge, concluded that the fatigue strength of riveted details depends strongly on the clamping state: tight details have high fatigue strength, details of indeterminate clamping state must be assessed more conservatively, and relaxed rivets require an even larger reduction of fatigue strength to ensure safety. Bertolesi et al. [19], through full-scale fatigue tests on a span and an upper cross girder dismantled from a riveted railway bridge built between 1913 and 1915, identified the fatigue life of the critical detail, the fatigue hot spots along the riveted element and the strain redistribution during crack growth, and calibrated on the measured results a numerical model used to estimate the remaining fatigue life by the S-N curve method of Eurocode 3.

2.4. Local deformation

Local deformation in steel girders arises from impact, overload, fabrication defects, or local instability due to corrosion reducing the web thickness [20]. Saad-Eldeen et al. [21], through experimental and numerical studies, showed that local deformation increases the stress concentration factor according to the relative deformation magnitude; for a deformation amplitude δ on a plate of thickness t within the range $\delta/t < 3$, the stress concentration factor can be preliminarily estimated as:

$$K_t \approx 1 + 0.5\frac{\delta}{t} \tag{5}$$

In addition to causing stress concentration, web thinning due to corrosion also increases the slenderness ratio and reduces the local buckling resistance of the web, which is an additional degradation mechanism to be considered for members under compression or shear. Recent Vietnamese experiments by Doan et al. [22] showed that initial geometric imperfections significantly change the lateral-torsional response and buckling resistance of hot-rolled I-beams.

2.5. Quantitative effect on fatigue life

Stress concentration is the bridge between geometric defects and the reduction of fatigue life. The effect of a defect on fatigue is characterised by the fatigue notch factor K_f, defined as the ratio between the fatigue limit of a defect-free specimen and that of a specimen containing the defect; the larger K_f is, the more strongly the defect reduces the fatigue strength [23]. The factor K_f is related to the elastic stress concentration factor K_t through the notch sensitivity factor q according to Peterson's formula [7,23]:

$$K_f = 1 + q\,(K_t - 1) \tag{6}$$

where q takes values from 0 to 1, reflecting the extent to which the material is actually affected by the defect compared with the theoretical elastic prediction. K_f is always smaller than or equal to K_t, because local plastic yielding at the defect root and the through-thickness stress-gradient effect reduce the actual influence of the defect compared with the theoretical elastic value [23]. For common structural steel, q is about 0.8 to 1.0 for defects with large radii, but smaller for sharp defects.

The relationship between the fatigue notch factor and fatigue life is derived from the S-N fatigue curve. According to EN 1993-1-9 [24] and steel bridge design practice, in the finite-life region this curve is a straight line on a log-log scale, with the equation $N \cdot (\Delta\sigma)^m = \text{const}$, in which the exponent m is the inverse slope of the curve. The value of m depends on the detail type and life region: $m = 3$ applies to the low-life region (fewer than 5×10^6 cycles), while $m = 5$ applies to the higher-life region (from 5×10^6 to 10^8 cycles) where the S-N curve has a gentler slope [24,25]. The fatigue curve equation is written as:

$$N_R \cdot (\Delta\sigma)^m = N_C \cdot (\Delta\sigma_C)^m = \text{const} \quad (7)$$

where N_R is the number of cycles to failure corresponding to a stress range $\Delta\sigma$, and $\Delta\sigma_C$ is the characteristic fatigue strength at $N_C = 2 \times 10^6$ cycles. When the local stress range at the defect location increases by the fatigue notch factor, that is $\Delta\sigma_{loc} = K_f \cdot \Delta\sigma_{nom}$, substituting into eq. (7) gives the ratio of residual fatigue life relative to the defect-free member, denoted η:

$$\eta = \frac{N_{def}}{N_0} = \left(\frac{\Delta\sigma_{nom}}{\Delta\sigma_{loc}}\right)^m = \frac{1}{K_f^m} \quad (8)$$

where N_{def} is the number of cycles to failure of the defective member, N_0 is that of the intact member, and the exponent m is the inverse slope of the S-N curve defined above. Eq. (8) shows that fatigue life decreases as the power m of the fatigue notch factor: for instance, with $m = 3$, when $K_f = 3$ the fatigue life is only $1/3^3 = 1/27 \approx 3.7$ % of that of the intact member.

The physical nature of this seemingly very large reduction must be clarified, since it is not a computational error but a direct consequence of the power-law relationship between stress and fatigue life. The S-N curve of steel has a power-law form: fatigue life is inversely proportional to the stress range raised to the power m. When a defect increases the local stress range by a factor K_f, fatigue life does not decrease linearly with K_f but with K_f^m. This is a fundamental characteristic of metal fatigue, confirmed experimentally in classical references [23,25], and is the reason why stress-concentrating defects, even at a purely local scale, can shorten fatigue life to only a small fraction of the original. Table 1 presents the residual fatigue life ratio η with $m = 3$, for the range of K_f values estimated from K_t with a notch sensitivity factor q in the range 0.8 to 1.0. It should be noted that these values are first-order estimates intended to indicate the trend and magnitude of the effect, and do not replace detailed fatigue analysis based on the actual stress spectrum and specific boundary conditions of each member.

Table 1. Estimated residual fatigue life ratio versus fatigue notch factor (with fatigue curve slope m = 3).

Defect type	$\boldsymbol{K_t}$	$\boldsymbol{K_f}$ **(with $\boldsymbol{q}$ from 0.8 to 1.0)**	**Fatigue life ratio $\boldsymbol{\eta = 1/K_f^m}$**
Localised corrosion (pitting)	1.2 to 3.0	1.2 to 3.0	0.04 to 0.58
Corrosion-induced perforation	2.5 to 3.0	2.2 to 3.0	0.04 to 0.09
Loose bolt or rivet	3.0 to 3.5	2.6 to 3.5	0.02 to 0.06

The results in Table 1 show that even accounting for K_f being smaller than K_t, defects causing high stress concentration still reduce fatigue life to only a small fraction of that of the intact member. Fig. 2 plots the relationship between the residual fatigue life ratio η and the fatigue notch factor K_f according to eq. (8) for two values of the inverse slope $m = 3$ and $m = 5$. These two values correspond to the two life regions of the S-N curve in EN 1993-1-9: $m = 3$ for the low-life region under large stress ranges, and $m = 5$ for the high-life region under smaller stress ranges. Of the two, $m = 3$ is the conservative value appropriate for most steel bridge details under large stress ranges, and is therefore used in the illustrative calculations of this paper. Fig. 2 reveals a feature deserving particular attention: the curve is very steep in the range of K_f from 1 to 2, meaning that even a moderate level of stress concentration is sufficient to cause a rapid drop in fatigue life. This is the core issue the paper wishes to emphasise: whereas the static load-carrying capacity decreases almost linearly with the degree of defect, fatigue life decreases as a power law, making fatigue a far more hazardous limit state that is nevertheless frequently overlooked in inspection practice.

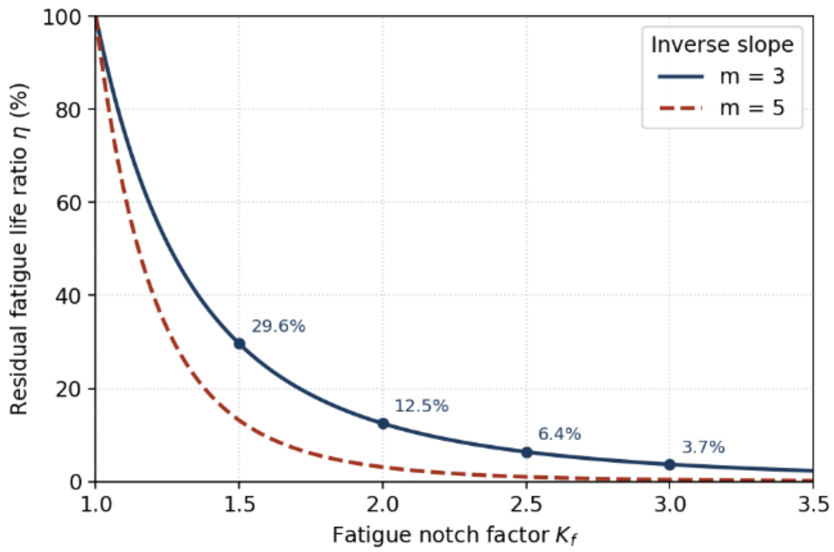

Figure 2. Relationship between the residual fatigue life ratio $\eta = 1/K_f^{\,m}$ and the fatigue notch factor K_f, for fatigue curve slopes $m = 3$ and $m = 5$.

2.6. Comparative summary of damage types

Table 2 summarises and compares the damage types according to the two degradation mechanisms and their effect on fatigue. Three main observations can be drawn. First, the stress concentration effect is the main governing factor in the reduction of load-carrying capacity for most local defects, with an influence much greater than area loss alone. Second, loose bolts and rivets are a particularly hazardous damage type because they cause no area loss but produce the highest stress concentration factor and therefore the strongest reduction of fatigue life. Third, damage types causing high stress concentration are also those that strongly reduce fatigue life.

Table 2. Comparison of damage types of steel girders in railway bridges.

Damage type	Area loss	K_t	Governing mechanism	Fatigue life reduction
Uniform corrosion	Medium to large	1.0 to 1.2	Area loss	Low
Localised corrosion (pitting)	Small	1.2 to 3.0	Stress concentration	Medium to high
Corrosion-induced perforation	Small	2.5 to 3.0	Stress concentration	High
Loose bolt or rivet	Negligible	3.0 to 3.5	Stress concentration	Very high
Local deformation	Small	2.0 to 2.5	Stress concentration	Medium to high

3. GAPS IN ASSESSMENT STANDARDS

3.1. The Vietnamese inspection standard TCVN 14478:2025

TCVN 14478:2025 (Inspection of road bridges) is the current national standard governing the inspection of ordinary road bridge structures, including steel bridges and steel-concrete composite bridges [26]. Although its direct scope is road bridges, the standard reflects the most recent and most systematic approach to steel structure defect assessment in the Vietnamese standard system and has high reference value for railway bridges, which are inspected under TCVN 11297:2016 [27] with a fundamentally similar approach. For new railway bridges, TCVN 13594-6:2023 [28] governs steel structures of 1435 mm gauge lines but, like the design standards in Section 3.2, does not address the residual capacity of members with corrosion- or hole-type defects.

The standard treats steel structure defects in a qualitative manner. Annex B lists typical defects including corrosion, distortion, gouges, dents, local instability, cracking, rupture, weak rivets, unfastened bolts and internal weld defects, and classifies the technical condition into four levels: Good, Average, Poor and Very Poor [26]. The threshold for transition to the Very Poor level for corrosion is described as a state in which the structure must be evaluated to determine the effect of the defect on strength; thus the standard triggers a requirement for quantitative assessment but does not provide the tool to perform it. When corrosion is present, the standard requires direct measurement of the degree of section reduction, that is, an area-loss-based approach that does not reflect stress concentration as analysed in Section 2.

Regarding load-carrying capacity assessment, according to Section 9 of the standard, the assessment is based on comparing measured data with design data and determining a load rating factor, corrected by a load-test factor obtained from the ratio of computed strain to measured strain [26]. This is an assessment at the level of nominal member stress. Regarding fatigue, the standard assesses through a stress spectrum obtained from vibration measurement combined with the linear cumulative damage rule, using nominal stress ranges and a limit number of cycles taken from the fatigue curve, with no mechanism to apply a fatigue notch factor K_f at local defect locations [26]. As a result, a member with a serious local defect may still be assessed as satisfactory for fatigue if the nominal stress is low, whereas according to the analysis in Section 2.5 the actual fatigue life at the defect location may have been seriously reduced.

Notably, the standard itself acknowledges the limitation of the vibration method: in Annex F, it states that the effect of deterioration or damage on vibration characteristics is generally small, and therefore recommends supplementing with local excitation methods to obtain information about damage [26]. The standard also recognises the role of stress concentration when it states that cracks usually occur at points of stress concentration at locations of abrupt section change, weld toes and rivet holes when rivets are weak [26]. However, these are qualitative guidelines, without accompanying stress concentration factor values, formulas or look-up tables to quantify the level of hazard.

3.2. Comparison with international standards

The international design standards show that the gap noted above is systematic rather than specific to the Vietnamese standard.

Eurocode 3 part EN 1993-1-5 [29] specifies the effective width method to assess the load-carrying capacity of thin plates, but does not address corrosion-induced perforations or local stress concentration effects. Part EN 1993-1-9 [24] specifies fatigue checking based on nominal stress with detail categories, each corresponding to an S-N curve of the form of eq. (7); however, these detail categories are established for standard machined bolt holes and do not include corrosion-induced perforations or loose bolts and rivets, and the standard also does not provide K_f values for such defects.

The AASHTO LRFD standard [30] and the AASHTO Manual for Bridge Evaluation [31] consider the effect of corrosion through a reduction of member thickness when determining resistance and the load rating factor, but do not provide formulas for the stress concentration factor due to perforations and lack guidance for loose bolts or rivets. The AISC 360 standard [32] determines the net section area by subtracting the hole area, but this provision applies only to standard bolt holes and does not consider the stress concentration effect. Table 3 summarises the gaps of the analysed standards.

Table 3. Gaps in current standards regarding the quantitative assessment of local defects.

Assessment criterion	**TCVN 14478:2025**	**EN 1993**	**AASHTO**	**AISC 360**
Qualitative defect identification	Yes	Yes	Yes	Yes
Measurement of corrosion area loss	Yes	Yes	Yes	Yes
Quantitative K_t for perforations	No	No	No	No
Quantitative K_t for localised corrosion	No	No	No	No
Criteria for loose bolts or rivets	No	No	No	No
Fatigue notch factor in fatigue assessment	No	No	No	No

4. ILLUSTRATIVE ANALYSIS ON AN ACTUAL STRUCTURE AND PROPOSED SUPPLEMENTS

4.1. Analysis of inspection data of a steel truss railway bridge under local conditions

To illustrate the findings presented in Sections 2 and 3, this section uses the actual inspection data of a steel truss bridge on the Thong Nhat railway line (designated Bridge A) as an analysis case. The values measured at the structure are recomputed under the local stress conditions at the defect location in order to compare with the results of the current assessment method. Bridge A is a steel truss bridge with high-strength bolted connections, built in 1979, comprising four truss spans with span lengths from 49.2 m to 72.2 m; at the time of inspection in 2019, the structure had been in service for 40 years.

The condition survey recorded damage types consistent with the classification in Section 2, including surface corrosion on the bottom chord with a thickness loss of about 10 to 15 %, through-thickness corrosion holes at gusset plates with diameters of about 15 to 25 mm, and a proportion of high-strength bolts with decayed heads that have lost their working capacity.

The load test was carried out using a loaded freight train in operation, with a load equivalent to about 63 % of the standard design load. The measured results and the assessment by the current method are summarised in Table 4, in which the maximum total stress recorded was 105.79 MPa at chord member X1 of span N3, reaching 56.8 % of the design strength of the steel ($R_u = 186$ MPa). Regarding vibration, the measured vertical natural frequency was 4.52 Hz compared with the value of 4.58 Hz computed from a model of the intact structure, corresponding to a change of $\Delta f/f \approx 1.3$ %, below the usual measurement error threshold of 2 to 3 %. On the basis of these results, the report concluded that the bridge satisfies the load-carrying capacity requirement and shows no significant stiffness reduction.

Table 4. Assessment results of Bridge A by the current method.

Assessment parameter	**Value**	**Criterion**	**Current conclusion**
Maximum nominal stress (member X1, span N3)	105.79 MPa	$R_u = 186$ MPa	Satisfactory (56.8 %)
Vibration frequency change $\Delta f/f$	1.3 %	2 to 3 % (error threshold)	No stiffness reduction

Re-analysing the above data under local stress conditions gives a significantly different result. At the location of the bolt that has lost its working capacity on member X1, the loss of the bolt clamping force shifts the load transfer from friction between the plates to bearing at the hole wall (the mechanism described in Section 2.3), so the connection hole approaches the free-hole state. The Kirsch solution gives $K_t = 3.0$ for a free circular hole (eq. (2)), and the global-local analyses of riveted railway connections by Imam et al. [2,17] give $K_t = 3.0$ to 3.5 for fully loose fasteners; $K_t = 3.0$ is adopted as the lower bound of this range. The following calculation is an illustrative estimate, since the inspection report does not provide the local geometry that a case-specific finite element analysis would require. Using the definition of the stress concentration factor in eq. (1), with the lower-bound value $K_t = 3.0$ and the measured nominal stress $\sigma_{nom} = 105.79$ MPa, the maximum local stress at the hole edge is determined:

$$\sigma_{max} = K_t \cdot \sigma_{nom} = 3.0 \times 105.79 \approx 317 \text{ MPa} \tag{9}$$

This local stress value corresponds to a ratio $\sigma_{max}/R_u \approx 1.71$ relative to the design strength $R_u = 186$ MPa. Three states must be distinguished here: (i) local yielding, where the material at the hole edge exceeds the elastic limit, exhausting the elastic safety margin and making the detail fatigue-critical; (ii) local resistance reduction, where repeated yielding and stress redistribution degrade the detail without impairing the member as a whole; (iii) member failure, governed by net-section plastic resistance or fracture, which the present data do not indicate. The result thus shows that the nominal-stress verification format loses validity at the defect location and local yielding has occurred there, not that the member has lost its load-carrying capacity. Over the published range $K_t = 3.0$ to 3.5, the local stress by eq. (9) is 317 to 370 MPa, so this conclusion holds over the entire range. The nominal-stress method thus overestimates safety at the local position, and the primary practical consequence is fatigue-related, namely crack initiation at the yielded hole edge under the very large number of railway load cycles, rather than static collapse. The actual local behaviour is beyond the scope of this review and will be quantified in a subsequent publication of the author by finite element models validated against laboratory beam tests under research project T2025-CT-010TD.

The effect on fatigue life is estimated as follows, with all assumptions stated explicitly. First, the fatigue notch factor is computed by eq. (6); with the stress concentration factor $K_t = 3.0$ and the notch sensitivity factor q in the range 0.8 to 1.0 for structural steel, the fatigue notch factor is in the range $K_f = 1 + q(K_t - 1) = 2.6$ to 3.0. Substituting into eq. (8) with the fatigue curve slope $m = 3$, the residual fatigue life ratio at the defect location relative to the intact member is $\eta = 1/{K_f}^m$, giving a result in the range 0.037 to 0.057, that is, the fatigue life is reduced to about 4 to 6 % of its original value, consistent with the range summarised in Table 1. Three remarks make the assumptions explicit. First, the notch sensitivity q = 0.8 to 1.0 is the range recommended for structural steel with millimetre-order notch radii [7,23]. Second, m = 3 is the finite-life slope of EN 1993-1-9 for fewer than 5×10^6 cycles, the conservative and governing region for railway loading (the sensitivity to m is shown in Fig. 2). Third, the estimated quantity is the relative ratio $\eta = N_{def}/N_0$, which for S-N curves of the same slope is independent of the detail-category constant (it cancels in the ratio), so no detail category needs to be assigned. The value of about 4 to 6 % is thus an illustrative first-order estimate, not a validated fatigue prediction, which would require the actual stress spectrum and a crack propagation analysis; over the full range K_t = 3.0 to 3.5 the ratio is about 2 to 6 %. The inspection report does not perform any fatigue assessment at the defect locations and therefore does not reflect this effect.

The difference between the two assessment approaches stems from the sensitivity of the measured quantities. The natural frequency is proportional to the square root of the ratio between the global stiffness and the mass of the structure; a local defect such as a single missing bolt or a small-diameter hole changes the global stiffness negligibly, resulting in a frequency change below the measurement error threshold, consistent with the value $\Delta f/f \approx 1.3\,\%$ recorded at the structure. Meanwhile, the load-carrying capacity and fatigue life at the defect location are governed by the local stress, which increases by the stress concentration factor. Therefore, local defects have little effect on global vibration quantities but a large effect on strength and fatigue, and this is the group of defects that global measurement methods have difficulty detecting. Data-driven monitoring of steel truss bridges is also developing in Vietnam, for example the damage identification framework of Pham et al. [33]; such global approaches complement, but do not replace, the local assessment discussed here.

For the case of Bridge A, neglecting the stress concentration effect leads to an overestimation of the load-carrying capacity and a failure to identify the fatigue life reduction at the location of the missing bolt. This analysis illustrates the gap noted in Section 3 and shows the need to supplement quantitative local assessment criteria, presented in Section 4.2.

4.2. Proposed supplementary assessment criteria

On the basis of the quantitative analyses in Section 2, the paper proposes a preliminary screening procedure that considers the stress concentration factor, applicable to plate-type details (flanges, webs, gusset plates) with hole-, pit- and deformation-type defects and based on idealised geometries from the published literature; it does not replace detailed finite element analysis or experimental assessment of actual connections. It comprises the following main contents.

First, supplementing a look-up table of stress concentration factors and screening factors for the various defect types to support rapid field assessment, with reference values compiled from published studies as presented in Table 5. The screening factor ϕ is determined by the inverse relationship with the stress concentration factor, $\phi = 1/K_t$, has an explicit mechanical meaning: local yielding initiates when the peak stress $\sigma_{max} = K_t \cdot \sigma_{nom}$ reaches the yield strength, that is, at a nominal stress f_y/K_t; hence $\varphi = 1/K_t$ is the ratio, relative to the intact detail, of the load level at first local yielding at the defect edge. This first-yield criterion is a conservative lower bound of the ultimate capacity, since after the onset of yielding the member retains reserve through plastic redistribution and its ultimate resistance is governed by the net-section plastic capacity. The factor φ is therefore proposed only as a preliminary screening criterion for prioritising inspection and intervention, relevant mainly to fatigue and serviceability, not for ultimate limit state verification, for which the net-section check remains governing. For a circular hole in a finite plate, eq. (3) may be used; for an elliptical hole, eq. (4).

Table 5. Proposed stress concentration factors and screening factors for defect types.

Defect type	K_t	**Screening factor** $\phi = 1/K_t$	**Reference**
Localised corrosion (pitting)	1.2 to 3.0	0.33 to 0.83	[9]
Corrosion-induced perforation	2.5 to 3.0	0.33 to 0.40	[7,15]
Loose bolt or rivet	3.0 to 3.5	0.29 to 0.33	[2,17]
Local deformation	2.0 to 2.5	0.40 to 0.50	[21]

Second, supplementing a fatigue check that considers the fatigue notch factor at the defect location. The nominal stress range is multiplied by the fatigue notch factor K_f determined by eq. (6) before being compared with the characteristic fatigue strength, equivalent to downgrading the fatigue detail category of the defect location. This approach quantifies the effect of the local defect on fatigue life according to eq. (8).

Third, regarding measurement methods, because the vibration measurement method has low sensitivity to local defects, the paper recommends combining vibration measurement for global assessment with local inspection methods such as strain measurement at suspected locations and load testing, in order to detect defects that the vibration method misses. This recommendation is consistent with the very orientation of TCVN 14478:2025 regarding the addition of local excitation.

Table 6 presents a framework of recommended actions according to the screening factor.

Table 6. Screening framework of recommended actions according to the level of damage.

Factor ϕ	**Level of damage**	**Recommended action**
Greater than 0.80	Minor	Periodic monitoring, no intervention required yet
0.50 to 0.80	Moderate	Enhanced monitoring, plan for repair
0.33 to 0.50	Serious	Early repair, possible load restriction
Less than 0.33	Very serious	Emergency repair or member replacement

The proposed stress concentration factors, screening factors and intervention thresholds are reference values compiled from the published literature under idealised geometry and loading conditions and are illustrated here on a single bridge; in practice the defect shape may be more complex and must be assessed by finite element analysis or measurement when high accuracy is required. They are therefore put forward as a screening framework for discussion and further development, not as generally validated criteria or a replacement for detailed analysis, and numerical and experimental validation is required before any normative use; for very serious cases, supplementary assessment is needed before a decision is made.

5. CONCLUSIONS

The paper has reviewed the common damage types of steel girders in railway bridges, quantitatively analysed their effects on load-carrying capacity and fatigue life, and clarified the gaps in current assessment standards regarding defective members. The main conclusions are as follows.

Regarding the degradation mechanisms and quantitative effects, a clear distinction must be made between cross-sectional area loss and local stress concentration. For common defects such as corrosion-induced perforations and connection holes when bolts or rivets become loose, stress concentration is the governing mechanism, with a stress concentration factor reaching 2.5 to 3.5, producing an effect two to three times greater than area loss. These defects reduce fatigue life to only a small fraction of that of the intact member, even when accounting for the fatigue notch factor being smaller than the elastic stress concentration factor.

Regarding the gaps in standards, the Vietnamese inspection standard TCVN 14478:2025 as well as the international standards Eurocode 3, AASHTO LRFD and AISC 360 all identify defects by qualitative condition ratings and assess them on the basis of area loss, but lack quantitative provisions for the stress concentration factor and for the effect of local defects on fatigue life. This is a systematic gap.

Regarding inspection practice, the analysis of data from an actual steel truss railway bridge shows that the current assessment method based on nominal stress and global vibration measurement does not fully reflect the working state at local defect locations. When recomputed under local conditions, the local stress at the location of the missing bolt exceeds the elastic limit of the steel, indicating local yielding and a fatigue-critical detail rather than loss of member capacity, whereas the current method concludes that the member works at a safe level; at the same time, the fatigue life at this location is reduced by about one to two orders of magnitude as an illustrative estimate, but is not assessed. Because local defects have little effect on global vibration quantities, they are difficult to detect by the vibration method, leading to the risk of overestimating the load-carrying capacity and degree of deterioration of the steel bridge.

Regarding the proposals, the paper recommends supplementing a look-up table of stress concentration factors, screening factors, a fatigue check considering the fatigue notch factor, and a framework of recommended actions for inspection work, and also recommends combining vibration measurement with local inspection methods. These proposals, compiled from the literature under idealised conditions and illustrated on a single bridge, are intended as a screening framework and require numerical and experimental validation before normative use. Future research will quantify precisely the reduction of resistance and fatigue life at defect locations by finite element analysis combined with experimental validation, in order to develop complete technical guidance for Vietnamese conditions. This validation, combining finite element quantification of stress concentration at hole- and corrosion-type defects on girder flanges with laboratory tests of steel beams, is being carried out under research project T2025-CT-010TD and will be reported separately.

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