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Evaluation of Shear Buckling Resistance for Numerical Model-Based Design of Slender Plated Structures

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Abstract

FEM-based design is getting widely used in the daily design practice of steel structures. This study investigates the influence of imperfections on the shear buckling resistance of structural members by using direct resistance check. A geometrically and materially nonlinear analysis with imperfections (GMNIA) is employed using a commonly utilized combination of geometric imperfection and residual stress. The numerical model is verified against a representative database of experimental tests to demonstrate its ability to accurately predict shear buckling capacity. To enable practical application, equivalent imperfections are assessed against the Eurocode buckling curve (EN 1993-1-5) and against the imperfection-residual stress combination. A statistical evaluation is then conducted to determine the model factor (γ_{FE}) associated with each equivalent imperfection. Based on the current findings, recommendations are provided for structural designers on the selection of appropriate imperfection magnitudes that comply with the safety requirements of Eurocode.

Keywords

Shear buckling, geometric imperfections, equivalent imperfections, structural safety, model factor.

1 Introduction

In recent decades, significant advancements have been made in the design of steel structures. The use of numerical model-based design is increasingly prevalent in civil engineering practice, facilitated by various methodologies. One approach is the indirect resistance check, also known as analysis requiring subsequent design checks. In this method, the numerical model is utilised to calculate the System Response Quantities (SRQs), which can then be employed to determine the structure's resistance using analytical design expressions. Alternatively, the direct resistance check involves using the numerical model to assess the load-carrying capacity of the structure directly. Although current design standards, such as EN 1993-1-5 [1], provide analytical and empirical-based design methods for evaluating shear buckling resistance, their accuracy has been debated for specific cases in the international literature [2,3] in the past. Modification proposals were also presented to solve the problems in [2] and [4].

Alternatively, geometrically and materially nonlinear analysis with imperfections (GMNIA) offers a highly accurate approach for determining shear buckling resistance, as demonstrated by numerous previous studies. However, geometric imperfections and residual stresses have a significant impact on the shear buckling resistance, particularly in slender plated structures. A primary challenge in applying GMNIA is selecting the appropriate shape and magnitude of imperfections and accurately modelling residual stresses, which often requires advanced modelling techniques. To simplify the process, equivalent geometric imperfections can be utilised for estimating buckling resistance. However, choosing an appropriate equivalent imperfection relies on a reliable estimate of the reference capacity for calibration. Additionally, experimental studies have revealed relatively large geometric imperfections in girder webs [5,6], highlighting the need to evaluate the validity of the commonly used equivalent imperfection value of $h_w/200$ specified in the new EN 1993-1-14 [7] standard. The main focus of the current study is to analyse the imperfection sensitivity of the shear buckling resistance and to recommend imperfection magnitudes for direct resistance check with accompanying model factor.

Therefore, this study investigates the application of GMNIA for the shear buckling resistance check and studies the equivalent geometric imperfections to be applied in the numerical model. The model is validated with a series of

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experimental results. The shear buckling resistance is determined using the validated numerical model and a widely accepted combination of geometric imperfections and residual stresses. The resistances derived from this combination, along with those obtained from EN 1993-1-5 [1], serve as reference resistances. Various equivalent geometric imperfections are analysed and compared independently to these reference resistances. Based on the comparison, the model factor (γ_{FE}) associated with the examined equivalent imperfections are established through statistical analysis and recommendations are developed for design practice.

2 Literature review

2.1 Shear buckling resistance models

In the international literature, many different shear buckling resistance models can be found. From these models, Höglund [8] developed the so-called rotated stress field model to provide an alternative solution for describing the shear post-buckling behaviour. The model was first applied to unstiffened plate girders and later extended to account for both transverse and longitudinal stiffeners. This model serves as the basis for the design method outlined in EN 1993-1-5:2006 [1]. It assumes that transverse membrane stresses in the web of girders with only end stiffeners are insignificant. To better match experimental observations, EN 1993-1-5:2006 [1] revised the χ_w factor to account for the effects of geometric imperfections and plastic resistance. A strain-hardening correction factor, η, was also incorporated to improve the model's predictive accuracy, with its value tailored to the steel grade to capture material nonlinearity more precisely. The shear buckling resistance must be verified if the height-to-thickness ratio (h_w/t_w) of the girder is larger than these limits: i) for unstiffened webs $\frac{h_w}{t_w} > 72\varepsilon/\eta$, and ii) for stiffened webs $\frac{h_w}{t_w} > 31\varepsilon\sqrt{k_\tau}/\eta$, where $\varepsilon = \sqrt{\frac{235}{f_y}}$, η depends on the steel grade, and k_τ is the shear buckling coefficient of the web. The ultimate shear buckling resistance, $V_{b,Rd}$, is determined by combining the web's shear buckling capacity, $V_{bw,Rd}$, with the contribution from the flanges $V_{bf,Rd}$, as shown in Eqs. (1-4). To take into account the interaction between the bending and shear, the provided method assumes that the presence of bending moments in the flanges reduces their ability to resist shear forces. Consequently, the shear buckling contribution of the flange is reduced by the ratio $\left(1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right)$.

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} < h_w t_w \frac{\eta f_{yw}}{\sqrt{3}\,\gamma_{M1}} \tag{1}$$

$$V_{bw,Rd} = \chi_w h_w t_w \frac{f_{yw}}{\sqrt{3}\gamma_{M1}} \tag{2}$$

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c\,\gamma_{M1}}\left(1-\left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right) \tag{3}$$

$$c = a \cdot 1.6 \cdot \left(\frac{b_f t_f^2 f_{yf}}{h_w^2 t_w f_{yw}}\right)^{0.44} \text{ but } c \leq 1.0 \tag{4}$$

Parameters h_w and t_w denote the web height and thickness, while b_f and t_f denote the flange width and thickness, respectively. The yield strengths of the web and flange are represented by f_{yw} and f_{yf}, respectively. The bending resistances M_{Ed} and $M_{f,Rd}$ correspond to the design bending moment and design plastic moment resistance of the flanges alone. The parameter c, which defines the position of the plastic hinges in the flanges measured from the transversal stiffeners or from the panel end, is calculated using Eq. (4), where a represents the spacing between transverse stiffeners. This equation has been changed from the original design equation of the EN 1993-1-5, as previous studies of Pedro et al. [2] and Bärnkopf et al. [4] proved that this equation leads to significantly better results and follows well the experimental and numerical simulation results. This analytical and empirical-based design method will be used as the reference resistance, cited as Eurocode-based resistance.

2.2 Definition of the model factor γ_{FE}

In the case of the direct resistance check, the consideration of the model uncertainties has special importance. Following the rules of EN 1993-1-14 [7], the inaccuracies and uncertainties of the numerical model and the applied analysis method are generally accounted for by using the model factor (γ_{FE}). The model factor covers both the uncertainties of the numerical model and the executed analysis type. However, it does not override the application of any other partial safety factors, which are specified for the given buckling check. It means the design uncertainties should be evaluated on the basis of the combination of the partial factors according to EN 1993 (all Parts) and the model factor (γ_{FE}).

In the case of a direct resistance check, the numerical model is intended to result in the characteristic value of the resistance. In this case, no analytical design resistance models are applied that could implicitly cover the uncertainties of the model. Therefore, irrespective of the analysed failure mode and model level, the uncertainties of the numerical model and the resistance calculation method need to be additionally considered. This can be done through the application of the model factor. In the numerical model-based design, one source of the uncertainties of the numerical model is the equivalent imperfections applied. The model factor can be used to quantify the uncertainty associated with each equivalent geometric imperfection, which is performed within the current study. According to EN 1993-1-14 [7], the model factor can be quantified through a comparison of the numerical calculation (R_{check}) to test results ($R_{test,known}$) or to known resistances obtained by approved methods ($R_{k,known}$).

$R_{k,known}$ is the calculated or known characteristic structural resistance,
$R_{test,known}$ is the known test result,
R_{check} is the computed resistance for the check structural resistance case.

The ratio ($\frac{R_{test,known}}{R_{check}}$ or $\frac{R_{k,known}}{R_{check}}$) should be determined for each case. The mean value (m_x) and the coefficient of variation (V_x) can be calculated for all the analysed ratios. The model factor (γ_{FE}) can be determined according to Eq. (5). The characteristic resistance can be calculated from the deterministic resistance according to Eq. (6).

$$\gamma_{FE} = \frac{1}{m_x\,(1-k_n V_x)} \text{ but } \gamma_{FE} \geq 1 \tag{5}$$

$$R_k = \frac{R_{GMNIA}}{\gamma_{FE}} \tag{6}$$

where, k_n is the characteristic fractile factor according to EN 1990:2021 [9], Annex D, Table D.1 (data row corresponding to V_X unknown should be used). In this study, the model factor is evaluated against the reference resistance ($R_{\mathrm{k,known}}$), obtained using advanced GMNIA. It is important to note that the model factor does not account for all sources of uncertainty; therefore, it should be used in combination with the partial safety factor γ_M.

3 Numerical model development and verification

A finite element model is developed in ANSYS [10] based on experimental test results available in the international literature. The model represents the test specimens and their loading and support conditions. Figure 1 shows a representative specimen. Four-node thin shell elements (Shell 181) are used to create the model. The mesh configuration, boundary conditions, and load application are illustrated in Figure 1 for one specific case.

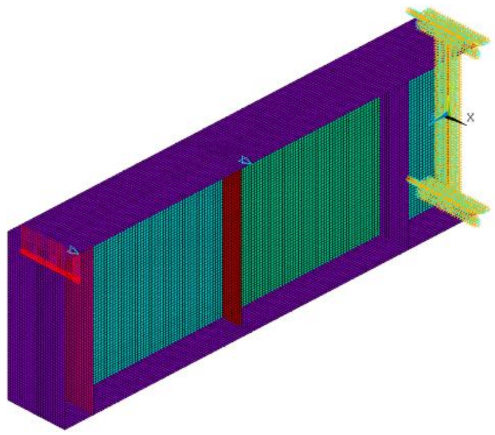

Figure 1 Finite element model.

The numerical analysis incorporates a quad-linear material model, which requires defining coefficients specific to the steel grades under investigation. These parameters are critical for representing the material's nonlinear response and include the yield strain ($\varepsilon_y = f_y/E$), the strain hardening modulus (E_{sh}) and the elongation coefficient (A), set to 0.2 to reflect post-fracture behaviour, used to impose a maximum limit to the value of ε_u. To prevent overestimation of material strength, a cut-off strain coefficient (C_1) is introduced, while (C_2) governs the slope of the strain-hardening modulus (E_{sh}). Table 1 summarises these coefficients in detail. The numerical model in this study incorporates geometric imperfections and residual stresses separately. Geometric imperfections are introduced based on a half-sine wave applied to the web of the girder. Whenever available, imperfection magnitudes are taken from the referenced studies; otherwise, the standard value of $h_w/1000$ is utilised [7]. Residual stresses are incorporated following EN 1993-1-14: 2024 [7] as shown in Figure 3. At the web-flange junctions, tensile residual stresses equal to the material yield strength are applied, while compressive stresses of 0.25 f_y are applied at the middle of the plates. The widths of the tensile and compressive zones are determined based on equilibrium equations.

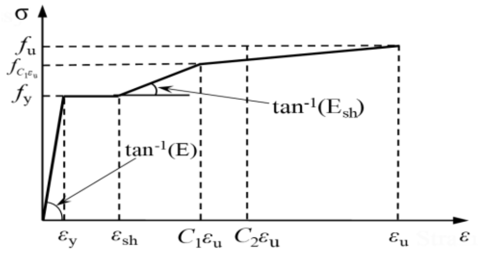

Figure 2 Material model for steel grades having a yield plateau according to EN 1993-1-14 [7].

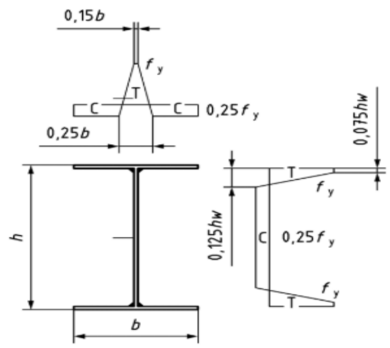

Figure 3 Residual stress pattern according to EN 1993-1-14 [7].

A mesh size equal to b/20, where b represents the plate width, provides a converged estimation of the shear buckling capacity. The validation process is presented in [4] in a more detailed manner. The numerical model is verified against experimental tests reported in [2,3]. One comparison of the failure mode obtained in the test and the numerical model is presented in Figure 4. The load capacities from both the laboratory tests and the developed numerical model show a close agreement within 5%, confirming the model's accuracy.

a)

b)

c)

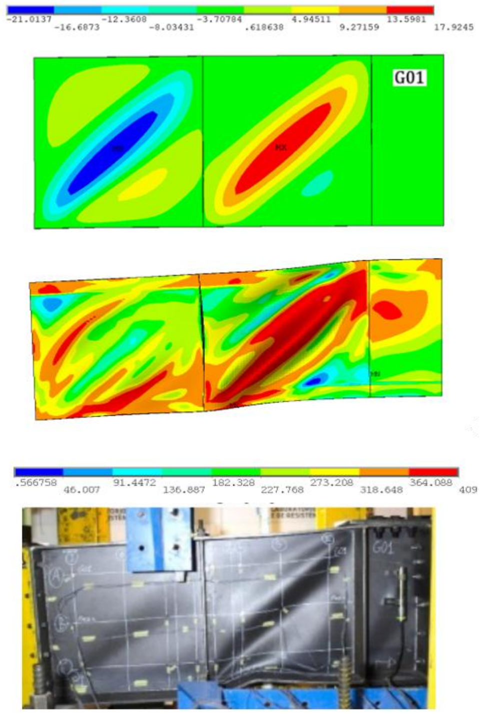

Figure 4 Model validation in the case of test specimen G01 [3], a) numerical model, transverse displacement at failure, b) numerical model, von Mises stresses at failure, c) failure in the laboratory test.

Table 1 Parameters of the applied material model.

	S235	S355	S460
f_y	235	355	460
f_u	360	510	540
ε_{sh}	0.010	0.015	0.030
ε_u	0.208	0.182	0.089
C_1	0.287	0.310	0.505
C_2	0.430	0.448	0.604
E_{sh}	1578	2310	3407
$C_1 \cdot \varepsilon_u$	0.060	0.057	0.045
$f_{C1\varepsilon u}$	313	451	510

4 Imperfection sensitivity analysis

A detailed numerical parametric study is performed to investigate the shear buckling resistance. The study examined various geometrical and material configurations to capture a range of practical geometrical scenarios. The flanges and web have yield strengths between 235 MPa and 700 MPa, covering multiple steel grades. Figure 5 shows the girder geometry along with the main dimensional parameters. The web thickness (t_w) ranges from 2.5 mm to 20 mm, and the web height (h_w) spans 400 mm to 1500 mm. Flange widths (b_f) vary between 50 mm and 550 mm, with flange thicknesses (t_f) of 7 mm to 60 mm, influencing both stiffness and load-carrying capacity. The cantilever length (a) ranges from 720 mm to 3400 mm, corresponding to web aspect ratios (α), which is defined as $\frac{a}{h_w}$, between 1 and 4. A detailed description of the parameters of the studied girders is shown in Table 2.

To evaluate how geometric imperfections affect shear buckling capacity, a sensitivity analysis is carried out. An illustrative case considers an I-girder with a 1000 mm web height (h_w), 250 mm flange width (b_f), and 16 mm flange thickness (t_f). The girder length (a) is equal to 1000 mm, leading to a a/h_w ratio of 1.0. The I-girder is composed of S235 structural steel, with web thickness ranging between 4 mm and 10 mm to investigate various slenderness ratios.

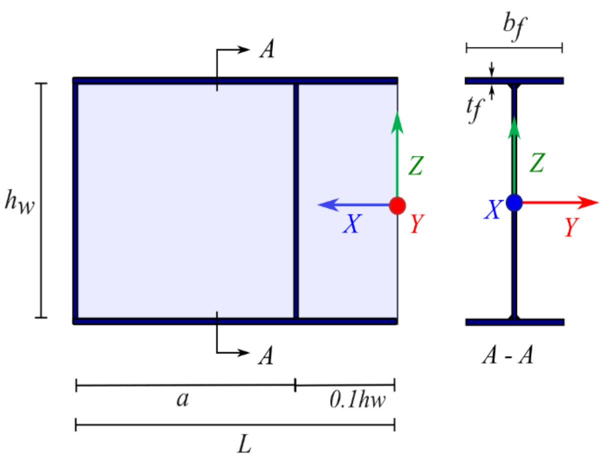

Figure 5 Geometrical properties of the I-girders.

Table 2 Material and Geometrical properties of the analysed cross-sections.

Property [unit]	Range
Yield strength of the flanges [MPa]	235 - 700
Yield strength of the web [MPa]	235 - 700
Web thickness t_w[mm]	2.5 - 20
Web height h_w [mm]	400 - 1500
Flange width b_f [mm]	50 - 550
Flange thickness t_f [mm]	7 - 60
Length a [mm]	720 - 3400
Web aspect ratio $\alpha = \frac{a}{h_w}$ [-]	1 - 4

Residual stresses are not included in this analysis in order to focus solely on the influence of geometric imperfections. Equivalent geometric imperfections are applied instead, with amplitudes varying from $h_w/25$ to $h_w/400$, enabling a thorough evaluation of their effect on shear buckling performance. The analysis results are presented in Figure 6, with the x-axis representing the equivalent imperfection factor (f, defined as h_w/f) and the y-axis showing the shear buckling resistance. The results indicate that geometric imperfections significantly affect shear buckling capacity, particularly for webs with small to moderate slenderness ($\bar{\lambda}_w < 1.56$). For girders with high slenderness, only exceptionally large imperfections noticeably affect the shear buckling capacity. Across the entire slenderness range, imperfections smaller than $h_w/200$ have a minimal impact, causing less than 2% resistance reduction compared to the maximum considered imperfection. By contrast, an imperfection of $h_w/25$ can reduce the buckling capacity by approximately 16% for medium-slenderness and 33% for high-slenderness ratios.

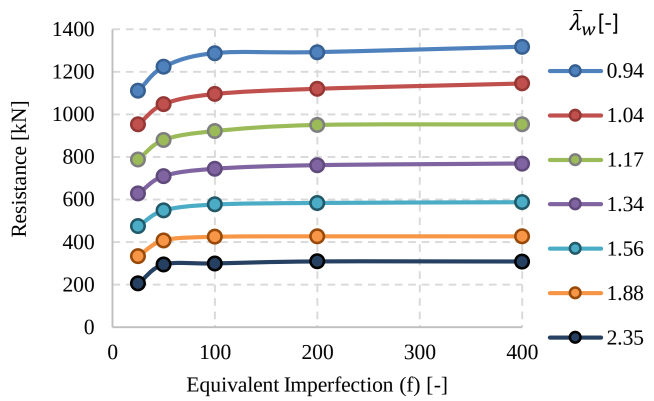

Figure 6 Imperfection influence on the buckling resistance for various web slenderness ratios.

Since the shear buckling capacity predicted by Eq. (1) has been widely criticised for its limited accuracy across different configurations, numerical simulations with a combination of residual stresses and geometric imperfections are also examined, and the results are compared to the Eurocode resistance to illustrate the shear buckling resistance predicted by each method. This approach enables the identification of an appropriate equivalent imperfection for reliably estimating shear buckling capacity. The outcomes of the analytical and numerical analyses are summarised in Figure 7, where the x-axis represents the web shear slenderness ($\bar{\lambda}_w$) and the y-axis represents the shear buckling reduction factor (χ). The black solid line represents the reduction factor according to EN 1993-1-5 [1], using *Eqs*. (1-2) representing rigid end post boundary conditions. Within the numerical model, geometric imperfections are applied in the form of a half sine wave along the web of the girder, using an amplitude of $h_w/1000$.

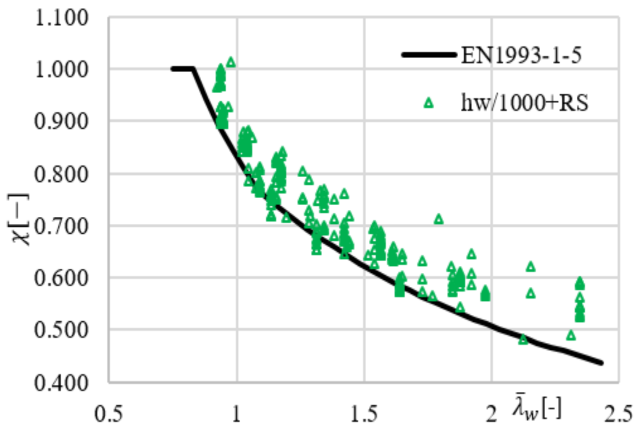

Figure 7 Comparison of shear buckling resistance predicted by the Eurocode and GMNIA analyses for various combinations of geometric imperfections and residual stresses.

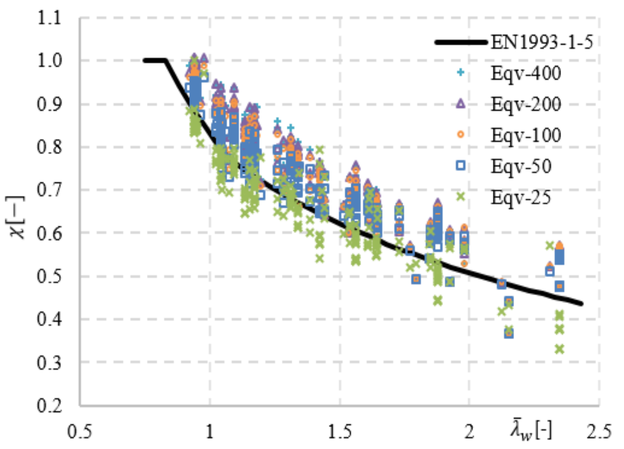

Figure 8 Comparison of shear buckling resistance for various equivalent geometric imperfection amplitudes.

It can be seen that the obtained numerical reduction factors show good agreement with the predicted values by EN 1993-1-5 [1], which represent a lower bound curve for the numerical results. It should be noted that the calculation method of *c* plays a significant role in this evaluation process; e.g. Eq. (4) has a large effect on the back-calculated resistances.

Various equivalent geometric imperfections are also examined to determine a reliable value for predicting shear buckling capacity by using equivalent geometric imperfections. The study considers imperfection amplitudes ranging from $h_w/25$ to $h_w/400$, enabling a thorough evaluation of their effects across different slenderness ratios and structural configurations. Figure 8 presents the results of the study. The analysis indicates that imperfection amplitudes smaller than $h_w/100$ consistently produce reduction factors that align with the EN 1993-1-5 design curve [1] (or lie above), which also represents a quasi-lower bound curve to the numerical results. Increasing the imperfection amplitude over $h_w/50$ causes some numerical results to fall beneath the Eurocode design curve, particularly for girders with medium slenderness. An amplitude of $h_w/25$ shows significantly lower resistances, with the corresponding results roughly balanced above and below the EN 1993-1-5 [1] shear buckling curve. Results also show that the difference between the resistances calculated using $h_w/100$ – $h_w/400$ is smaller than 2%. The calculation results using $h_w/200$ are presented in Figure 9.

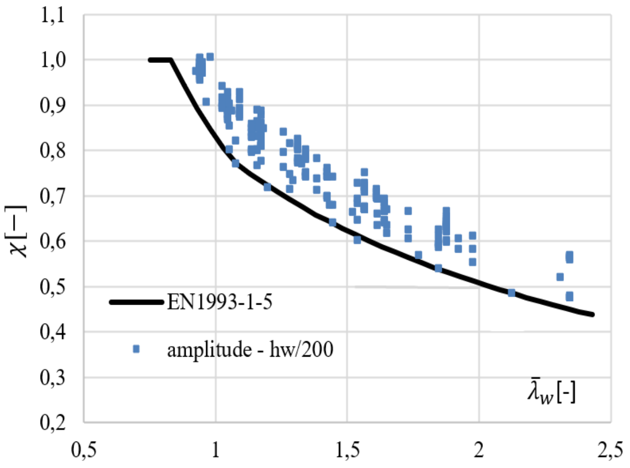

Figure 9 Comparison of shear buckling curve and numerical results by using equivalent geometric imperfection with $h_w/200$ amplitude.

The diagram shows that the Eurocode-based shear buckling curve gives a lower bound curve to the numerical results, consistent with the database obtained using $h_w/1000$ geometric imperfection plus residual stresses. This is more or less clear and evident, as the shear buckling resistance curve is based on test results and numerical simulations using $h_w/200$ as equivalent geometric imperfections.

Direct comparison of the resistances is presented in Figure 10. The horizontal axis shows the computed resistance by using geometric imperfections and residual stresses ($h_w/1000$+RS); the vertical axis shows the resistance computed by using equivalent geometric imperfection with a magnitude of $h_w/200$.

Similar evaluations are made for all geometric imperfection magnitudes analysed, and the relevant databases are statistically evaluated. Table 3 presents the results of the statistical evaluation. Results show that the obtained results using $h_w/200$ give the best approach, which represents the accurate buckling resistance with a mean value of 1.0 and a coefficient of variation of 0.045.

To statistically evaluate the obtained database, the model factors are calculated for all equivalent geometric imperfection magnitudes; for the $h_w/200$, its value is equal to 1.08. Although the model factor for $h_w/50$ is smaller due to the larger mean value 1.02, the $h_w/200$ represents a better value to estimate the buckling resistance due to the smaller coefficient of variation. This is clearly illustrated by the results presented in Figure 10 and summarized in Table 3, which together provide a detailed comparison of the observed behavior and corresponding numerical values.

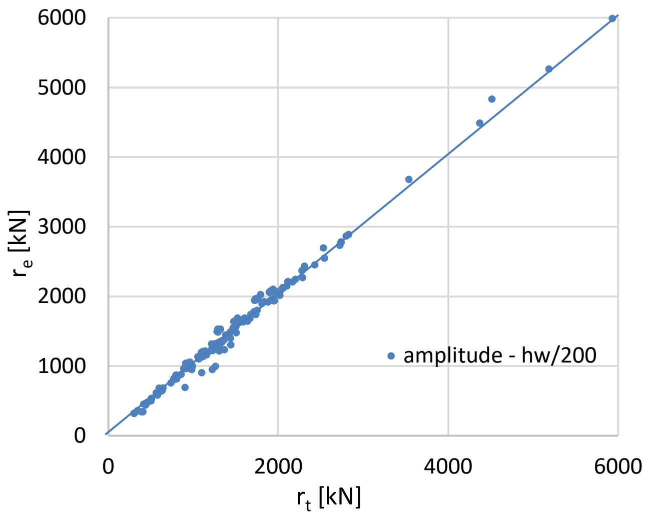

Figure 10 Comparison of shear buckling resistances by using equivalent geometric imperfection of h_w/200 and geometric imperfections and residual stresses (h_w/1000+RS).

Table 3 Statistical measures for the studied equivalent imperfections.

Measure	hw/400	hw/200	hw/100	hw/50	hw/25
Mean	0.986	1.000	1.010	1.020	1.116
$\boldsymbol{\sigma}$	0.048	0.045	0.050	0.050	0.116
CoV	0.049	0.045	0.050	0.050	0.104
kn 5%	1.64	1.64	1.64	1.64	1.64
$\boldsymbol{\gamma_{FE}}$	1.10	1.08	1.08	1.07	1.08

5 Conclusion

The current study examined the effect of geometric imperfections and residual stresses on the shear buckling capacity of girders using advanced numerical modelling combined with reliability-based assessment techniques. A commonly used combination of geometrical imperfections accompanied by residual stresses was evaluated through GMNIA simulations and compared with the Eurocode design curve. Consequently, several equivalent geometric imperfections are evaluated by comparing the numerical results with the reference resistance.

Results demonstrate that geometric imperfections profoundly affect shear buckling capacity, especially in slender webs. Smaller imperfection amplitudes consistently align with or even exceed Eurocode design curve predictions. However, larger imperfections, such as h_w/50 or h_w/25, can significantly reduce the shear buckling capacity compared to the Eurocode-based reference resistance.

The research conducted statistical evaluations to determine appropriate model factors for each equivalent imperfection, ensuring compliance with Eurocode safety requirements. The study finds that an imperfection amplitude of h_w/200 provides a good balance, yielding a mean resistance value of 1.0 relative to GMNIA simulations and an associated model factor (γ_{FE}) of 1.08.

It should also be noted that further statistical investigations could be made on the obtained database, as the value of γ_{M1} also contains part of the model uncertainties, so the model factor evaluated in combination with the γ_{M1} could be slightly further adjusted.

Acknowledgements

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