



Improved shear buckling resistance model for hybrid I-girders with slender web

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ABSTRACT

The present study focuses on examining the shear buckling behavior and the interaction between shear and bending (M-V interaction) in longitudinally unstiffened girders with slender webs. The applicability of the existing shear buckling resistance model, given in EN 1993-1-5, is evaluated through comprehensive numerical analysis. Based on the results, an enhanced design equation is proposed to improve accuracy, optimize safety, and cost-efficiency in the design of hybrid girders. The parametric numerical investigation indicates that the contribution of the flange to shear buckling resistance does not conform to the current EN 1993-1-5 rules. Specifically, for girders with lightweight flanges, the existing approach underestimates the flange's contribution, leading to overly conservative resistances. Conversely, for I-girders with heavy flanges, the current method may produce unsafe predictions. Moreover, the study identifies inconsistencies and inaccuracies in the present design equation for hybrid girders, showing that the current resistance model does not accurately reflect the shear capacity of hybrid configurations. Therefore, a new, refined design formula is proposed to more accurately estimate the flange contribution, aligning closely with numerical simulation results.

1. Introduction

Welded plate girders have been a cornerstone of steel construction for decades, especially in long-span bridges and steel industrial buildings, where structural efficiency and material optimization are important. These girders rely on their slender webs to carry shear forces, while the flanges primarily resist bending moments. However, shear buckling remains a significant design challenge for girders with slender webs. Modern design standards recognize the post-buckling strength of webs, which continues to contribute significantly to shear capacity even after elastic buckling, largely due to tension field action. Recent advances in the analysis of shear buckling behavior have highlighted the role of flanges in the shear buckling resistance. Historically, the focus was primarily on the web; however, both experimental and numerical studies have demonstrated that flanges can play a stabilizing role, especially in girders with heavy flanges that influence the development of the post-buckling stress field. Despite these insights, current design standards still face difficulties in accurately predicting shear buckling resistance across different girder configurations. This is largely because existing formulas for the flanges' contribution are empirical, calibrated on a limited set of tests with specialized configurations.

To improve the reliability of these predictions, the underlying mechanical models need to be revised to better reflect the observed behavior from experimental tests and advanced numerical simulations. Building on this foundation, the present study investigates how flange contribution affects the shear buckling capacity of hybrid steel plate girders through a comprehensive parametric analysis. Using advanced geometrically and materially nonlinear analysis with imperfections (GMNIA), the research explores a wide range of geometric configurations, with a particular focus on the stabilizing influence of flanges on shear resistance and post-buckling response. The outcomes of this study are expected to lead to refined design recommendations that more accurately account for flange-web interaction, thereby enhancing the predictive accuracy of shear buckling resistance models currently used in structural design practices. Therefore, the current research aims to investigate the contribution of flanges for a wide range of geometric configurations, with particular emphasis on their influence on the shear buckling resistance and post-buckling response, especially for hybrid girders.

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2. Literature review

2.1. Review of previous studies on shear buckling

The concept of post-buckling behavior was first recognized by Wilson in 1886 [1], with Wagner [2] introducing the first design criteria in aeronautical engineering in 1931. Research on the ultimate resistance of steel plate girders for civil engineering applications gained momentum in the 1960s, led by contributions from Basler [3–5] and Rockey et al. [6,7]. Basler [3–5] conducted experimental investigations into the stress distribution of the tension field in plate girders with transverse stiffeners. He proposed that plate girder flanges were too flexible to fully anchor the tension field, leading to the assumption that the buckling shear stress (τ_{cr}) persisted throughout the web plate even after elastic buckling. His model proved reliable for plate girders with slender webs and weak flanges, particularly with aspect ratios around 3. Research performed by Rockey and Skaloud [7–9] at Cardiff University built upon Basler's work by demonstrating that flanges could contribute to post-buckling resistance. The Cardiff model proposed that the tension field was anchored to both stiffeners and flanges, with plastic hinges forming at flange-to-stiffener connections. It suggested that ultimate failure resulted from a combination of plastic hinges in the flanges and web yielding [6]. While both Basler and Cardiff models incorporated shear force contributions (V_{pc}) from diagonal tension fields along with the elastic shear buckling force (V_{cr}), they tended to be conservative for girders with large web panel aspect ratios due to limited tension field contributions. Höglund [10] introduced the rotated stress field model, initially for unstiffened plate girders and later adapted for transverse and longitudinal stiffeners. This model, later incorporated into EN 1993-1-5: 2006 [11], assumed that transverse membrane stresses in the web were negligible for long beams with only end transverse stiffeners. Subsequent refinements in EN 1993-1-5: 2006 [11] adjusted X_w values to account for experimental deviations caused by imperfections and plastic buckling. The introduction of the η factor compensated for strain-hardening effects, varying with the steel grade. Several researchers have identified overestimations and underestimations in existing models, depending on the web panel aspect ratio. Yoo and Lee [12] showed that sufficiently stiff webs could redistribute shear loads through compression stresses, reducing dependence on stiffeners. Studies by Glassman and Garlock [13] highlighted that buckled webs retained compressive resistance perpendicular to the tension field, affecting post-buckling behavior. Augustyn et al. [14] conducted a parametric study to evaluate the contribution of flanges and transverse stiffeners in the web post-buckling behavior of I-shaped steel girder. It was found that increasing the flange thickness in a given girder configuration can alter the post-buckling behavior at the end of the first stage of post-buckling, i.e. instead of gradual unloading; the girder may exhibit positive hardening, shifting the ultimate shear resistance from the end of stage I to stage II, which results in a larger displacement and 10–15 % increase in shear resistance. Alinia et al. [15] conducted a parametric study to determine the origin of plastic hinges that emerge in the experimental tests. The study concluded that hinges are caused by shear deformations near the supports and not by the bending stresses induced by the tension field. Also, the hinge location and the width of the tension field are both related to the rigidity of the end stiffener and flange dimensions. Biscaya et al. [16] conducted a numerical parametric study, revealing that EN 1993-1-5 [11] yielded conservative shear resistance estimates for small flange sizes but potentially unsafe results for large flange-to-web area ratios. They proposed revised equations to modify the c/a ratio, enhancing shear resistance predictions. Freitas et al. [17] conducted full-scale load tests on transversely stiffened plate girders, measuring internal forces in intermediate transverse stiffeners. Their findings revealed that EN 1993-1-5 [11] stiffener design forces were highly conservative, with experimental values averaging 70 % of the standard design force. Nascimento et al. [18] similarly tested eight plate girders, confirming that transverse stiffeners were often oversized. Pedro

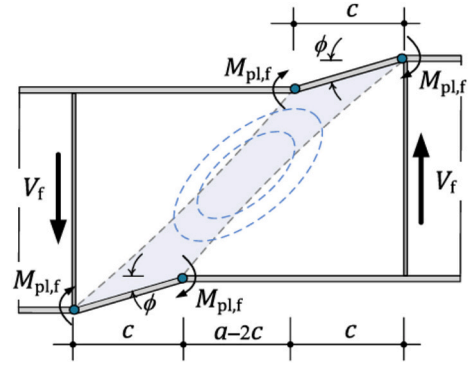


Fig. 1. Failure mechanism and physical meaning of the EN 1993-1-5 shear buckling resistance model.

et al. [19] compiled a database of 261 experimental tests on steel plate girders loaded in shear. The authors proposed modifications to more accurately predict the ultimate resistance and better reflect the contribution of the flanges by proposing a new modified value of the distance c in the flange contribution formula. The authors showed that further improvements could be made to reduce the scatter of the resistance estimated by EN 1993-1-5 [11]. In recent years, several studies have investigated the use of deep neural networks to address buckling problems in steel structures, highlighting a broader trend toward incorporating advanced machine learning techniques into structural stability analysis [20–24]. Elamary et al. [25] utilized deep neural networks to estimate the shear buckling resistance of steel girders. This study developed a regression learner model using neural networks trained on 100 experimental results to predict the shear strength of steel beams. The proposed model was validated against experimental and theoretical data.

2.2. Shear buckling according to EN 1993-1-5

The shear buckling capacity must be evaluated when the web height-to-thickness ratio (h_w/t_w) exceeds the following thresholds: i) for unstiffened webs $\frac{h_w}{t_w} > 72\epsilon/\eta$, ii) for stiffened webs $\frac{h_w}{t_w} > 31\epsilon\sqrt{k_r}/\eta$, where $\epsilon = \sqrt{\frac{235}{f_y}}$, $\eta = 1.2$ for steel grades up to S460 and $\eta = 1$ for higher steel grades and k_r is the shear buckling coefficient of the web. According to EN 1993-1-5, the ultimate shear resistance $V_{b,Rd}$ is the sum of the contribution of the web shear buckling resistance $V_{bw,Rd}$ and the contribution of the flange $V_{bf,Rd}$, as presented in Eq.(1–3).

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

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

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

$$c = a \left(0.25 + 1.6 \frac{b_f t_f^2 f_{yf}}{h_w^2 t_w f_{yw}} \right) \quad (4)$$

where, h_w is the web height, t_w is the web thickness, b_f is the flange width, t_f is the flange thickness, γ_{M1} is a partial safety factor equals to 1.0. f_{yw} and f_{yf} are the yield strength of the web and the flanges, respectively. M_{Ed} is the design value of the bending moment resistance, $M_{f,Rd}$ represents the design plastic moment of resistance of the cross-section, considering only the effective area of the flanges. The parameter c defines the location of the plastic hinges that develop in the flanges and is

Table 1
Dimensions of the test specimens [18].

Girder	a/h_w [–]	a [mm]	r [mm]	L [mm]	h_w [mm]	t_w [mm]	b_f [mm]	t_f [mm]
G01	1	830	355	2015	830	4	300	15
G02	1	830	355	2015	830	4	300	15
G03	1	830	355	2015	830	4	300	15
G04	1	830	355	2015	830	4	300	15
G05	1	830	355	2015	830	4	300	15
G06	1	830	355	2015	830	4	300	15
G07	1.5	1245	525	3015	830	4	300	15
G08	1.5	1245	525	3015	830	4	300	15

given by Eq.(4), where a is the distance between transverse stiffeners. The mechanical model used by the EN 1993-1-5 for the flange contribution is presented in Fig. 1.

2.3. Executed research strategy

The main aim of the current investigation is to develop an improved version of the flange contribution to the shear buckling resistance model, which has been proven inaccurate by previous investigations. The current paper investigates this resistance from the design point of view of hybrid girders. This means that special focus is given to the yield strength ratio of the flange and web plate. So, the study investigates the shear buckling behavior of hybrid steel girders, focusing on the contribution of the flanges and the accuracy of existing design formulas. A geometrically and materially nonlinear model with imperfections (GMNIA) is employed to capture the complex nonlinear behavior associated with shear buckling. This approach enables a more accurate estimation of buckling resistance compared to simplified analytical or linear methods, as it accounts for the interaction between geometric imperfections, material yielding, the post-buckling effect, and structural instability. Despite inherent approximations in modelling material behavior, geometric imperfections, and residual stress patterns, GMNIA has consistently demonstrated its effectiveness in accurately predicting buckling resistance, making it a trusted tool in structural stability analysis. A parametric study is conducted using a validated finite element model, where a single panel of the girder is analyzed to simplify computations and analysis. The girder is modeled with a rigid end post, and the load is applied through a master node to ensure dominant shear behavior. Residual stresses and imperfections are incorporated based on standardized values available in EN 1993-1-5 [11] and prEN 1993-1-14 [26]. A broad range of girder configurations is analyzed to reflect the varying responses observed under shear, ensuring the robustness of the proposed model.

The study program is performed according to the following steps:

- 1- Validation of the numerical model against shear buckling experimental test.
- 2- Conducting a parametric study to investigate the effect of the plastic hinges and the width of the tension field on the buckling capacity.
- 3- Evaluating the shear buckling capacity based on the measured distance for the plastic hinges.
- 4- Conducting a parametric study on a wide range of configurations to investigate the contribution of flanges to the buckling capacity.
- 5- Developing an analytical-based formula to enhance the accuracy of flange contribution estimation.
- 6- Conducting a comparison against the currently available formulas and applying statistical measures to check the dispersion of the currently available formulas.

3. Numerical model development and verification

3.1. Description of the experiment used for the validation

The experiment used for the validation of the numerical model is carried out at the Structures and Strength of Materials Laboratory at the Instituto Superior Técnico – University of Lisbon, and is published in detail by Nascimento et al. [18]. A total of 8 large-scale thin-walled girders were tested, they were subjected to shear force. The nominal values of the geometrical data of the specimens tested are summarized in Table 1 using the notations given in Fig. 2.

The same geometry was used for each specimen, except that two different stiffener types are applied, and the specimens also differ in the a/h_w ratio, as shown in Table 1. The applied stiffener type and the geometry of them are summarized in Table 2. The single-sided intermediate transverse web stiffeners adopted are illustrated in Fig. 3, showing two different configurations: (a) rectangular flat plates and (b) tee

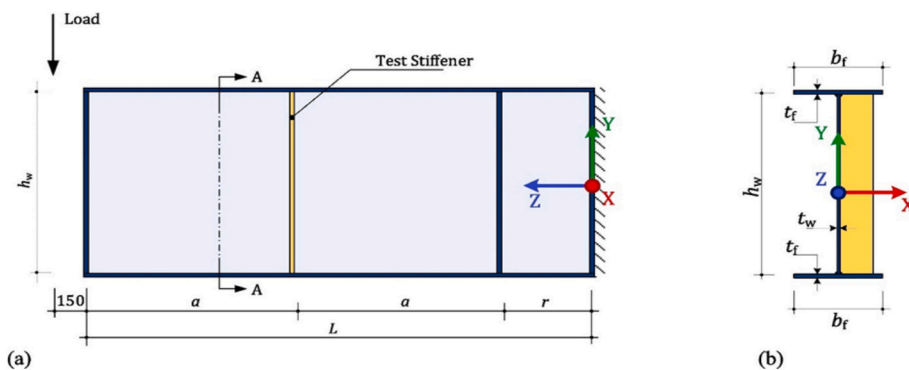


Fig. 2. Geometry of plate girder specimens; (a) Side view of the test specimens, (b) cross-section and notations [18].

Table 2
Geometry of transverse stiffener and allocation to each specimen [18].

Girder	Flat stiffener	Tee stiffener	b_{st} [mm]	t_{st} [mm]	$b_{st,f}$ [mm]	$t_{st,f}$ [mm]	$h_{st,1}$ [mm]	$h_{st,2}$ [mm]
G01	●	○	100	10	–	–	830	–
G02	●	○	100	10	–	–	830	–
G03	○	●	100	4	100	6	700	80
G04	○	●	100	4	100	6	700	80
G05	●	○	100	10	–	–	830	–
G06	○	●	100	4	100	6	700	80
G07	●	○	100	10	–	–	830	–
G08	○	●	100	4	100	6	700	80

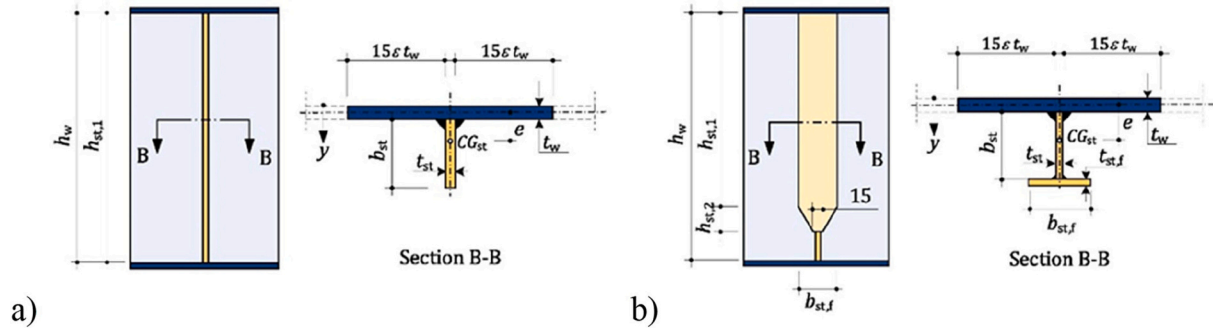


Fig. 3. Geometry of plate girder stiffeners: (a) flat stiffener, (b) Tee stiffener [18].

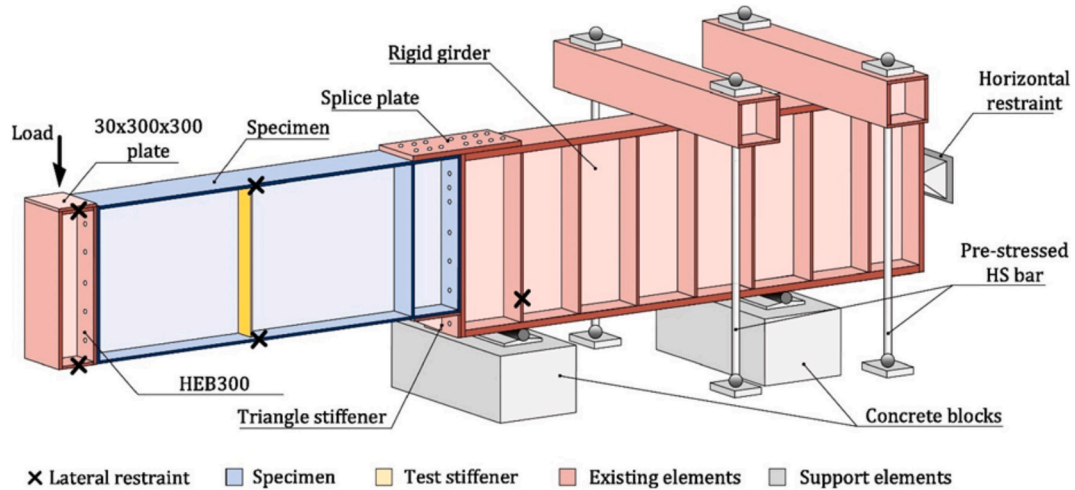


Fig. 4. General test setup from [18].

sections.

During the test setup, the test specimen is bolted to other load members through the end plates. The general test setup is shown in Fig. 4. Prior to the experimental tests, each test specimen was subjected to detailed imperfection measurements to accurately determine the actual shape and imperfections. The mechanical properties of steel were assessed using tensile tests on steel coupons using a universal testing machine and a high-precision optical extensometer to certify the material reliability, which is summarized in Table 3.

Additional experimental results from four independent research programs reported in the international literature were collected to validate the accuracy and robustness of the proposed numerical model. A summary of these tests, including key parameters, is provided in Table 4.

3.2. Numerical model development

Based on the laboratory tests, a numerical model is developed using Ansys 19.2 finite element software [30]. The model is validated by checking the mathematical and geometrical adequacy of the finite elements and by the test results. The numerical model used in this paper contains only the test specimen and the stiff transition element that was attached to the specimen tip, according to Fig. 4. The girder is subjected to shear force at this stiff transition element. For the force application, nodes along a crossline are chosen. The developed model is a full shell model using four-node thin shell elements (Shell 181), as shown in Fig. 5, together with the finite element mesh, boundary, and loading conditions.

Table 3
Main measured material mechanical characteristics [18].

Girder	Coupon location	Thickness [mm]	f_y [MPa]	f_u [MPa]	ϵ_u [-]	E [GPa]
G01 to G06	Web (G01 + G02) and tee stiffeners web	4	381	531	0.15	211
	Web (G03 to G06)	4	380	523	0.15	224
	Tee stiffeners flange	6	394	501	0.17	226
	Flat stiffener	10	389	518	0.14	175
	Flanges	15	370	488	0.17	194
G07 to G08	Web and tee stiffener web	4	383	499	0.17	190
	Tee stiffener flange	6	389	525	0.14	193
	Flat stiffener	10	419	549	0.17	202
	Flanges	15	340	488	0.17	217

Table 4
Dimensions of the test specimens.

Girder	a/hw [-]	a [mm]	hw [mm]	tw [mm]	bf [mm]	tf [mm]
G6-T1 [4]	1.50	1905	1270	4.9	308	19.8
G6-T2 [4]	0.75	953	1270	4.9	308	19.8
G6-T3 [4]	0.50	635	1270	4.9	308	19.8
G7-T1 [4]	1.00	1270	1270	5	310	19.5
H1T2 [27]	1.50	1905	1270	10	459	24.8
H2T1 [27]	1.00	1270	1270	9.9	459	51.2
C-AC2 [28]	5.45	2490	457.2	3.1	102	9.7
B [28]	1.00	1200	1200	4.5	240	12
S-2 [29]	1.82	581	319	3.2	100	10.5
S-3 [29]	1.21	577	477	3.2	101	10.5

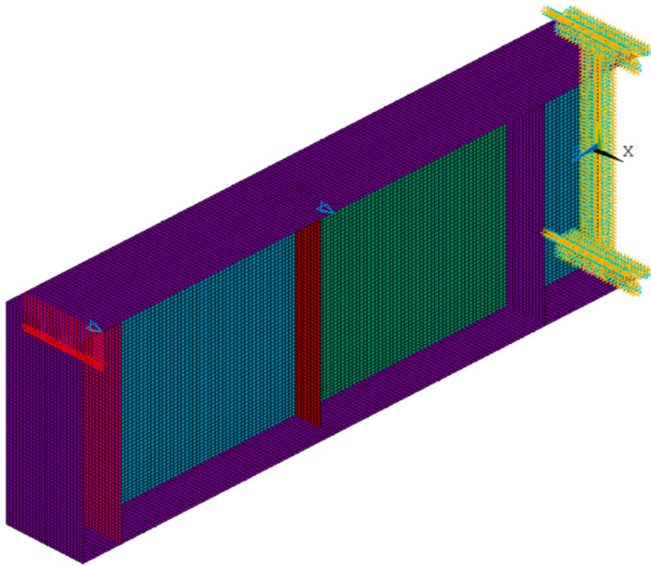


Fig. 5. Finite element model (mesh, boundary, and loading conditions).

3.3. Applied material model

The applied material model is based on the recommendations of the international literature. A quad-linear material model is adopted to accurately represent the nonlinear behavior of normal-strength steel (NSS), conforming to the provisions outlined in prEN 1993-1-14 [26]. The graphical representation of the model is provided in Fig. 6, while its mathematical formulation is detailed in Eqs.(5)-(10), following the work of Yun and Gardner [31]. This material model is defined from three basic parameters – elastic modulus (E), yield strength (f_y), and tensile strength (f_u). Except to the calculations using the measured material data, elastic

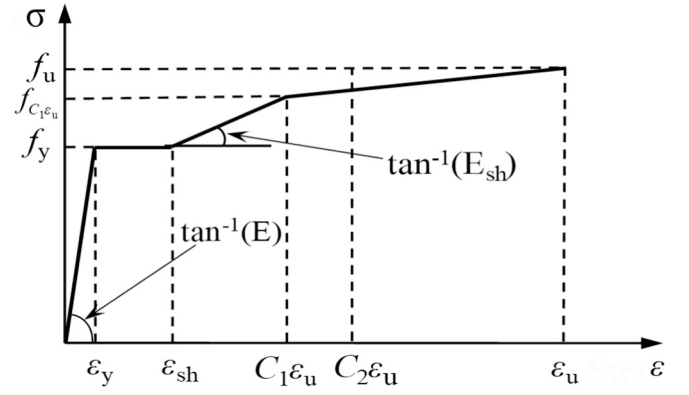


Fig. 6. Material model for Normal Strength Steel (NSS), according to prEN 1993-1-14 [26].

Table 5
Parameters of the applied material model.

Parameter	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
$C_1 \cdot \epsilon_u$	0.060	0.057	0.045
$f_{C_1\epsilon_u}$	313	451	510

modulus of 210 GPa with a Poisson's ratio of 0.3 is used.

$$\sigma(\epsilon) = \begin{cases} E\epsilon, & \epsilon \leq \epsilon_y \\ f_y, & \epsilon_y < \epsilon \leq \epsilon_{sh} \\ f_y + E_{sh}(\epsilon - \epsilon_{sh}), & \epsilon_{sh} < \epsilon \leq C_1 \epsilon_u \\ f_y C_1 \epsilon_u + \frac{f_u - f_y C_1 \epsilon_u}{(\epsilon_u - C_1 \epsilon_u)}(\epsilon - C_1 \epsilon_u), & C_1 \epsilon_u < \epsilon \leq \epsilon_u \end{cases} \quad (5)$$

$$E_{sh} = \frac{f_u - f_y}{C_2 \epsilon_u - \epsilon_{sh}} \quad (6)$$

$$\epsilon_{sh} = 0.1 \frac{f_y}{f_u} - 0.055 \text{ but } 0.015 \leq \epsilon_{sh} \leq 0.03 \quad (7)$$

$$\epsilon_u = 0.6 \left(1 - \frac{f_y}{f_u} \right), \text{ but } 0.06 \leq \epsilon_u < A \quad (8)$$

$$C_1 = \frac{\epsilon_{sh} + 0.25(\epsilon_u - \epsilon_{sh})}{\epsilon_u} \quad (9)$$

$$C_2 = \frac{\epsilon_{sh} + 0.4(\epsilon_u - \epsilon_{sh})}{\epsilon_u} \quad (10)$$

The implementation of the quad-linear material model in the numerical model requires the determination of specific coefficients tailored to the steel grades under investigation. These coefficients are critical in accurately defining the material response and include key parameters such as the yield strain ($\epsilon_y = f_y/E$), the strain hardening modulus E_{sh} , and the elongation coefficient (A), which is set to 0.2 to account for post-fracture elongation. Additionally, the cut-off strain coefficient (C_1) is introduced to prevent an overestimation of strength, while C_2 , governs the slope of strain hardening (E_{sh}). A comprehensive summary of these coefficients is presented in Table 5.

For high-strength steel (HSS) materials, the numerical simulations

Table 6
Material properties for HSS.

Steel grade	Yield strength (f_y) (MPa)	Ultimate strength (f_u) (MPa)
S500	500	625
S690	700	850
S960	960	1115

Table 7
Highest web imperfection amplitudes reported [18].

Girder	Left panel [mm]		Right panel [mm]	
	Max.	Min	Max.	Min
G01	+5.51	-7.13	hw /116	+4.04 -7.26
G02	+5.14	-11.80	hw /70	+9.17 -0.28
G03	+10.49	-4.54	hw /79	+3.78 -5.66
G04	+5.06	-5.36	hw /155	+7.49 -1.50
G05	+11.16	-3.06	hw /74	+6.78 -4.73
G06	+3.51	-8.31	hw /100	+2.48 -5.66
G07	+10.71	-2.76	hw /78	+5.40 -6.89
G08	+1.48	-10.48	hw /79	+10.71 -4.30

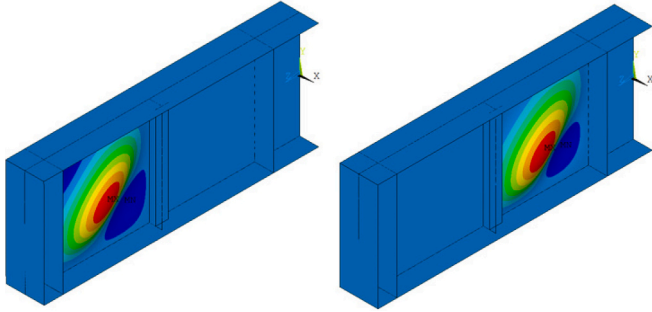


Fig. 7. Imperfection shape in the numerical model.

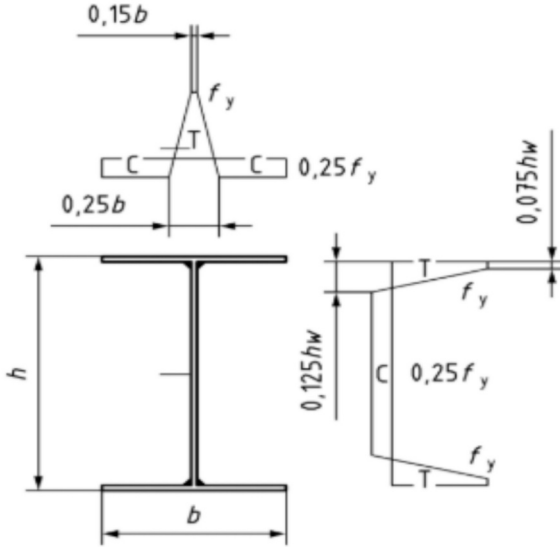


Fig. 8. The utilized residual stress model.

employ the Ramberg-Osgood-type material model, a nonlinear elastic-plastic formulation that incorporates strain hardening, as given in Eq. (11). The Material properties for HSS are shown in Table 6.

$$\varepsilon = \frac{\sigma}{E} + 0.002 \left(\frac{\sigma}{f_y} \right)^n \quad (11)$$

Table 8
validation of the numerical model.

	R_{exp} [kN]	R_{num} [kN]	difference
G01 [18]	510,6	507,6	0,59 %
G02 [18]	505,8	507,6	-0,36 %
G03 [18]	509,9	510,5	-0,12 %
G04 [18]	502,5	510,5	-1,57 %
G05 [18]	523,8	523,1	0,14 %
G06 [18]	519,9	512,1	1,52 %
G07 [18]	419,9	417,0	0,70 %
G08 [18]	409,4	415,8	-1,55 %
G6-T1 [4]	516	515.12	0.2 %
G6-T2 [4]	662	668.23	-0.9 %
G6-T3 [4]	787	820.29	-4.2 %
G7-T1 [4]	623	589.22	5.4 %
H1T2 [27]	3421	3535.31	-3.3 %
H2T1 [27]	4079	3916.41	4.0 %
C-AC2 [28]	120	113.73	5.2 %
B [28]	760	749.41	1.4 %
S-2 [29]	161	159.41	1.0 %
S-3 [29]	198	195.75	1.1 %

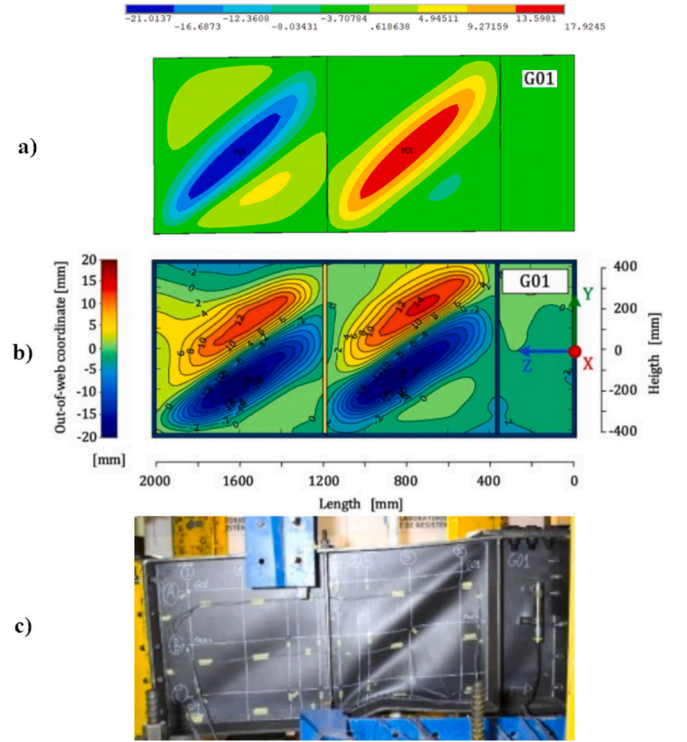


Fig. 9. Model validation in the case of G01, (a) numerical model, transverse displacement at failure, (b) transverse displacement at failure according to Nascimento et al. [18], and (c) failure in the laboratory test.

3.4. The applied imperfections

In this paper, the effects of geometric imperfections and residual stresses are considered separately in the numerical model. The shape of the geometric imperfections is taken to be the first eigenmode magnified to the amplitudes given in Table 7, illustrated in Fig. 7.

Fig. 8 shows the residual stresses are applied according to the prEN 1993-1-14: 2024 [26], so at the web-flange junction are tension-type stresses equal to the yield strength. At the middle of the plates, compressive residual stresses are applied with a pressure value of $0.25 \cdot f_y$ (the width of the tensioned and compressed parts is calculated by equilibrium conditions).

The residual stresses and the measured geometric imperfections are given by Nascimento et al. [18], so they are used in the numerical

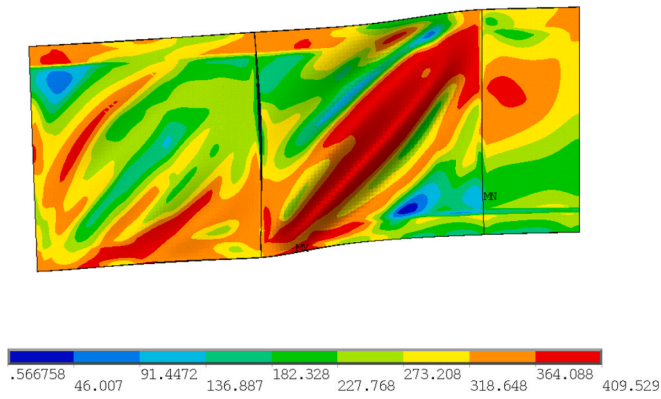


Fig. 10. Failure mode in the numerical model.

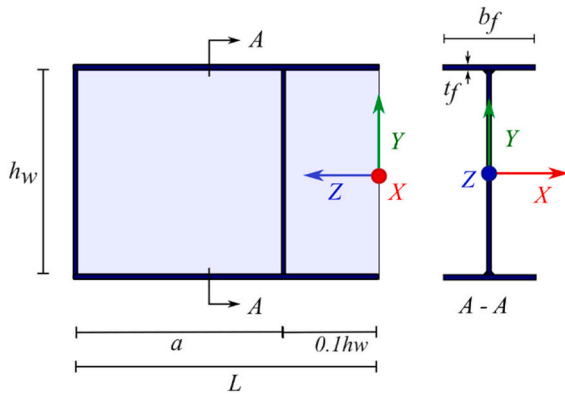


Fig. 11. Graphical representation of the geometrical properties of the I-girders.

validation process by using the measured data. For the validation of the other tests collected from the international literature and for the numerical analyses performed later in this study, standardized imperfection values according to EN 1993-1-14 [26] are applied, where an imperfection of $h_w/1000$ is accompanied by the residual stress pattern shown in Fig. 8.

3.5. Validation of the numerical model

The load capacities obtained in the laboratory experiments and using the developed numerical model are shown in Table 8. It can be seen that the results are within 2 %, which proves the accuracy of the developed numerical model for the tests obtained from Nascimento et al. [18]. However, for the experimental results sourced from international literature with limited available data, the maximum discrepancy between the numerical and experimental results remains below 6 %, indicating acceptable agreement despite the data constraints. Figure 9 shows the failure mode shape based on the FE model, the numerical model according to Nascimento et al. [18], and the laboratory test results. It can be seen that the maximum transverse displacement shows good agreement, and the failure mode is the same. Figure 10 shows the Von Mises stress diagram at the ultimate load level, which refers to shear buckling behavior.

4. Investigation of the shear buckling behavior

4.1. Parameters used in the numerical study

After the detailed model validation, a wide-ranging numerical parametric study is executed to check the shear buckling resistance of I-girder beams. The I-girders under investigation have varying dimensions

Table 9

Geometrical properties of the analyzed 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 α [–]	1–4

and material properties, covering a wide range of structural configurations. The yield strength of both the flanges and the web varies between 235 MPa and 700 MPa, accommodating different steel grades. Fig. 11 shows a graphical representation of the geometrical properties of the girder. The web thickness (t_w) ranges from 2.5 mm to 20 mm, while the web height (h_w) spans from 400 mm to 1500 mm. The flange width (b_f) varies from 50 mm to 550 mm, and the flange thickness (t_f) ranges from 7 mm to 60 mm, influencing the overall girder stiffness and load-bearing capacity. The length of the cantilever part of the girder (a) extends from 720 mm to 3400 mm, resulting in a web aspect ratio (α) between 1 and 4. These parameters define the geometric and material characteristics of the I-girder, affecting its structural performance and stability. Detailed characteristics of the girders examined are presented in Table 9.

4.2. Flange contribution in hybrid steel girders

The performance of hybrid steel girders is significantly influenced by the contributions of both the web and flange. In this section, the role of the flange in enhancing the structural capacity, stability, and overall behavior of hybrid steel girders is examined. The validated numerical model is used to perform the parametric study. However, to simplify the complexity of the analysis, only one panel of the girder is modeled, where a rigid end post is modeled at one side, while the load is applied to a master node on the opposite side. To ensure symmetrical behavior, the master node is restrained against rotation in the X and Y directions and against translation in the X direction. Geometric imperfections with an amplitude of $h_w/1000$ are introduced in the model, together with the residual stress distribution depicted in Fig. 8. Both imperfections and residual stresses are incorporated consistently in the analyses to accurately simulate the structural behavior [26,32].

As shear stresses develop, they induce diagonal tension and compression within the web, potentially leading to buckling along these stress paths if left unsupported. The flanges at the top and bottom provide both in-plane and out-of-plane restraint, disrupting the buckling mode shape and encouraging more stable deformation patterns. This interaction is especially critical in slender webs where shear buckling tends to govern the failure response. However, if the flanges are also slender or lack stiffness, they may deform along with the web, diminishing their restraining effect and possibly participating in the buckling mode. In such scenarios, the web and flange deformations interact in a nonlinear manner. Several researchers observed that the flange contribution comes from the formation of plastic hinges in the flanges at the sections where the diagonal tensile stress field is anchored, where stronger flanges with larger bending moment resistance have a larger contribution to the shear capacity [19]. To investigate this observation, the yield strength of the flanges is increased while maintaining the yield strength of the web constant for several configurations of the I-girder, as shown in Table 9. Several f_{yf}/f_{yw} ratios are investigated, including 1.0, 1.5, 2.0, and 3.0. An example of the performed study is shown in Fig. 12, where the I-girder has a web height of 1300 mm, a web thickness of 12 mm, a flange width of 400 mm, a flange thickness of 25 mm, and a length-to-height ratio $\alpha = \frac{a}{h_w} = 1.0$. The web is made of S235 steel and the flanges are made of S235, S355, S460 and S690, corresponding approximately to the mentioned f_{yf}/f_{yw} (ratios)

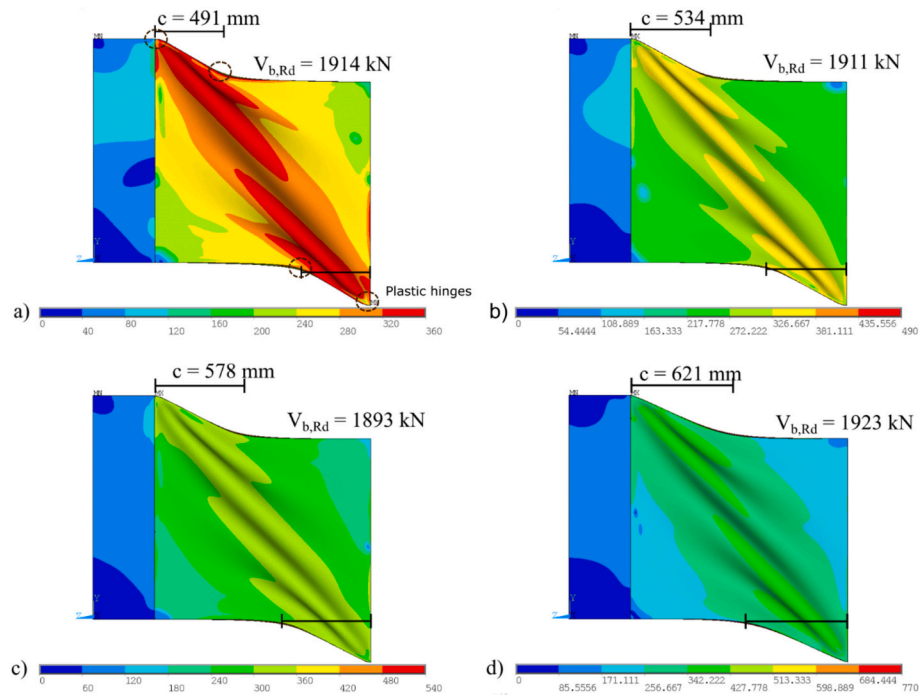


Fig. 12. Effect of the yield strength of the flanges on the tension field for the following ratios: a) $\frac{f_{yf}}{f_{yw}} = 1.0$, b) $\frac{f_{yf}}{f_{yw}} = 1.5$, c) $\frac{f_{yf}}{f_{yw}} = 2.0$, d) $\frac{f_{yf}}{f_{yw}} = 3.0$.

Table 10

Comparison between the analytical buckling resistance according to EN 1993-1-5 and the numerical analysis.

$\frac{f_{yf}}{f_{yw}}$	EN 1993-1-5			Numerical Analysis		
	$V_{bf,Rd}$ [kN]	$V_{bw,Rd}$ [kN]	$V_{b,Rd}$ [kN]	$V_{bf,Rd}$ [kN]	$V_{bw,Rd}$ [kN]	$V_{b,Rd}$ [kN]
1.0	60.5233	1853.6	1914.12	149.17	1765.15	1914.32
1.5	175.943	1853.6	2029.54	155.89	1754.99	1910.88
2.0	256.524	1853.6	2110.12	147.21	1756.15	1903.36
3.0	398.704	1853.6	2116.57	152.63	1770.86	1923.49

As shown in Fig. 12, the value of c increases with the increase of the stiffness of the flanges compared to the stiffness of the web, where for $\frac{f_{yf}}{f_{yw}} = 1.0$, the c value is equal to 491 mm, while for $\frac{f_{yf}}{f_{yw}} = 3.0$, the c value is equal to 621 mm, indicating an enlarged tension field. Additionally, Fig. 12 shows the numerical shear buckling resistance for each case. It is possible to notice that there is a small increase in the buckling resistance. However, the increase in the resistance is relatively small and does not agree with the EN 1993-1-5 formula for estimating the buckling resistance, as shown in Table 10.

The forces in the flanges and the web for the analytical resistance are calculated according to Eq.(2–3), while the numerical resistances are determined using numerical integration of stresses in the numerical model. Numerical results show, the flange contribution $V_{bf,Rd}$, remains approximately constant and is not influenced by the f_{yf}/f_{yw} ratio. However, in the analytical approach, this contribution depends on both the f_{yf}/f_{yw} ratio and the c value. Consequently, the total buckling capacity of the girder stays relatively similar, with minor dependence on the stiffness of the flanges.

The results of the parametric study for the configurations shown in Table 9 are presented in Fig. 13. The x-axis represents the f_{yf}/f_{yw} ratio, while the y-axis shows the measured c/a values for various girder configurations with α values of 1.0, 1.5, and 2.0. From the results, it can be observed that the c/a values follow a parabolic increasing trend as the f_{yf}/f_{yw} ratio increases, regardless of the α value. However, this effect does not significantly influence the girder's overall capacity, as the

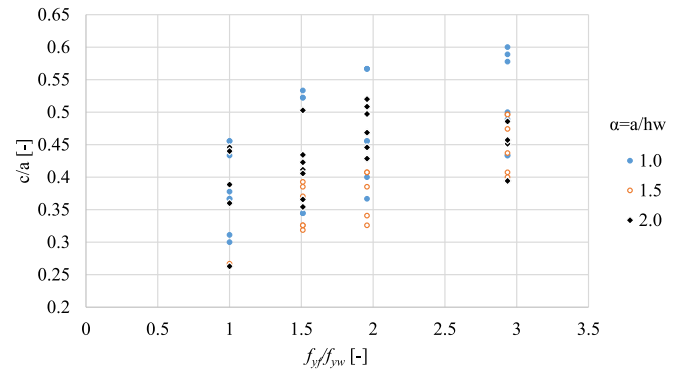


Fig. 13. The measured c/a values for a wide range of configurations.

buckling resistance remains nearly the same across the different f_{yf}/f_{yw} ratio. This indicates that while flange size affects the distribution of internal forces, it has a small impact on the ultimate buckling strength of the girder.

This observation leads to the fact that the shear buckling resistance and especially the flange contribution will be highly overestimated for hybrid girders by using the standard design method (acc. to Eqs. (1)–(4)), which is not confirmed by the numerical results. The reason behind this is that the yield strength of the flange is considered in the flange contribution according to Eq. (3) with a linear trend. However, the value of c does not increase linearly with the hybridity ratio, defined as the yield strength of the flanges to the yield strength of the web. This leads to an increase in the flange contribution to the shear buckling resistance in hybrid girders. However, this resistance increase is not proved by the numerical results for hybrid girders. Even if the latest proposal of Pedro et al. (19) is considered within the resistance model of the Eurocode (according to Eq. (12)), whose equation is considered the best fit proposal for non-hybrid girders, the problem is not diminished.

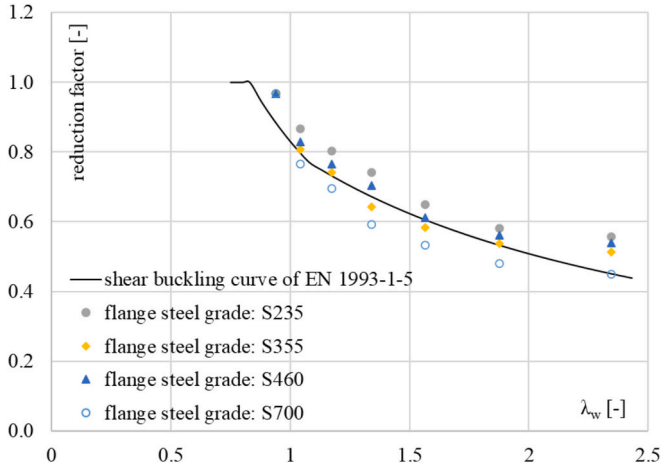


Fig. 14. Shear buckling reduction factor using c values taken from Eq. (12).

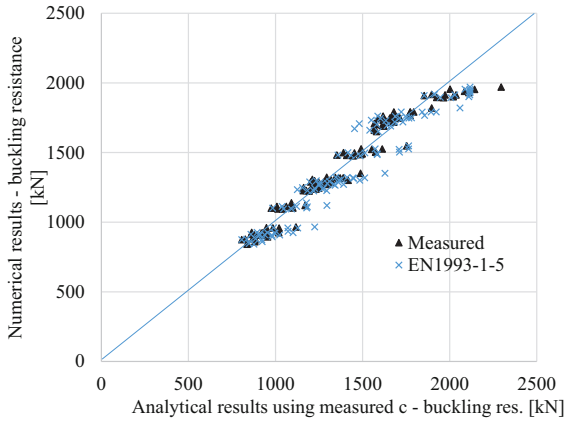


Fig. 15. Comparison of the buckling capacity using both the measured c values and the values obtained from Eq.(4).

$$c = a \cdot 1,6 \cdot \left(\frac{b_f t_f^2 f_{yf}}{h_w^2 t_w f_{yw}} \right)^{0,44} \# \quad (12)$$

Figure 14 illustrates the back-calculated shear buckling reduction factors in comparison to the current shear buckling resistance curve specified in EN 1993-1-5 for the same girders, using different steel grades for the flanges. The horizontal axis represents the relative slenderness ratio related to shear buckling, as defined by EN 1993-1-5. The vertical axis indicates the buckling reduction factor to be applied in the calculation of web shear buckling resistance. The results demonstrate that all numerical calculations for non-hybrid girders lie above the buckling curve, validating the accuracy of the standard design equation for non-hybrid girders. However, the findings also reveal that increasing the flange yield strength results in unsafe levels of shear buckling resistance for hybrid girders, highlighting the necessity for an improved resistance model for these cases.

To illustrate the impact of the measured values of c on buckling resistance, Fig. 15 presents a comparison between the analytical resistance calculated using both the measured c values from the numerical analysis and those calculated by Eq.(4) against the resistance obtained directly from the numerical analysis.

This comparison highlights potential discrepancies between the formula in Eq. (4) and the numerical results, offering insights into the accuracy and applicability of Eq. (4) for estimating the buckling capacity of hybrid steel girders. The results also indicate that the buckling capacity would not be perfectly predicted even if the exact (measured)

Table 11

Statistical measures of the ratio of the numerical results to the EN1993-1-5 analytical resistance using the measured c and Eq.(4) values.

Type/Statistical Measure	Min	Max	σ	μ	CoV
Using measured c values	0.858	1.128	0.055	1.009	0.054
Using Eq. (4)	0.788	1.153	0.073	0.977	0.075

value of the parameter c were used in the current EN 1993-1-5 design model. This suggests that accurately determining c alone would not fully resolve the limitations of the shear buckling resistance model. While it might improve predictions for hybrid girders, it would still lead to results that are sometimes conservative and other times unsafe. A statistical evaluation of over 800 numerical simulations for the shear buckling resistance is presented in Table 11. When using measured c values, the mean ratio of numerical resistance to calculated resistance is 1.009, with a standard deviation of 0.055. In contrast, using Eq. (4), the mean ratio drops to 0.977 with a standard deviation of 0.073, indicating a slight overestimation of the shear buckling resistance and, consequently, a risk of unsafe design outcomes.

The flange contribution to shear buckling demonstrates a complex physical interaction with the web, particularly in the post-buckling stage. After the development of the post-buckling of the web, the web alone cannot sustain additional shear resistance, and a decline in capacity follows. However, this reduction is moderated by the flanges and stiffeners, which continue to engage through tension-field action and hardening effects. The flanges sustain part of the shear force due to axial forces induced by bending, contributing to a plateau or even a secondary peak in shear resistance [19]. Experimental and numerical observations show that a plastic mechanism tends to form in the flanges, driven by the tension field between them. This mechanism does not act independently of the web, as the interaction between web and flanges provides mutual restraint. Thus, the flange's contribution cannot be treated in isolation, and its influence is typically underestimated by simplified theoretical models. Flanges play a key role in stabilizing the structure post-buckling, delaying failure by resisting part of the shear force and enhancing the overall ductility of the web-flange system. Numerical results also showed that for most of the analyzed cases, four plastic hinges developed in the top and bottom flanges, as shown in Fig. 12. However, the tension field is also extended to the transverse stiffeners, allowing the formation of plastic hinges in the stiffeners, which might counteract the plastic hinges in the flanges. By using this observation, it is possible to develop the formula in Eq. (14), which is based on $2 \cdot M_{pl,f}$, where $M_{pl,f}$ is defined according to Eq. (13). The formula mainly consists of two parts: the first depends on the relative geometrical stiffness of the flange to the stiffness of the web, and the second part is the relative yield strength of the flanges to the web. The background of the shear buckling resistance models is well documented by Pedro et al. in [19], which derivation perfectly fits the currently proposed design equations:

$$M_{pl,f} = \frac{b_f t_f^2}{4} f_{yf} \# \quad (13)$$

$$\frac{c}{a} = 2 \cdot \sqrt{\frac{b_f \cdot t_f^2}{h_w^2 \cdot t_w}} \cdot \left(\frac{f_{yf}}{f_{yw}} \right) \# \quad (14)$$

$$V_{bf,Rd} = \frac{2 \cdot M_{pl,f}}{c \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) \# \quad (15)$$

To test the applicability of the proposed formula, a total of 800 numerical tests are performed for the configurations shown in Table 9. The numerical results are then compared to the analytical resistances according to Eq. (1–3) by using the proposed formula for c in Eq. (14). The comparison is shown in Fig. 16, where the x-axis shows the resistance using Eq. (14) and the y-axis shows the numerical analysis. It is possible to see that the provided formula accurately estimates the shear buckling

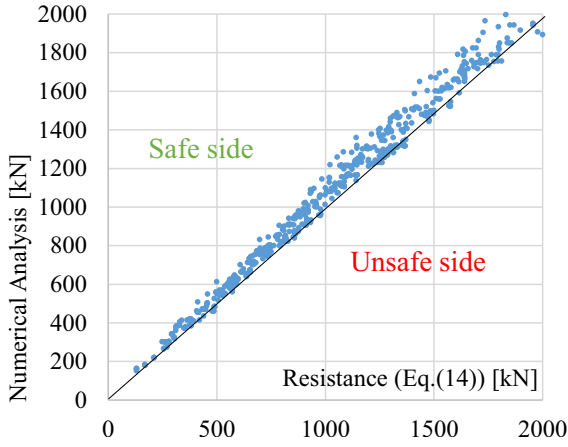


Fig. 16. Comparison between the shear buckling resistances using (Eq.(14)) and the numerical analysis.

resistance, with approximately all results on the safe side.

Another way to visualize results is shown in Fig. 17, where the x-axis shows the shear buckling slenderness $\bar{\lambda}_w$ and the y-axis presents the buckling reduction factor. The reduction factors are calculated for the web of the girders and based on the total buckling capacity obtained from the numerical results subtracting the contribution of the flanges, which is calculated according to Eq. (14). It is possible to observe that all the points lie either on or above the shear buckling curve defined by the EN 1993-1-5 [32], indicating safe-side results for the entire analyzed parameter range. It also means that for the analyzed cases, the flange contribution is not overestimated by the resistance model using Eqs. (14)–(15).

Fig. 18 presents a comparison between the total numerical resistance and the total analytical resistance estimated by the different available formulas for estimating the c value including the EN 1993-1-5 [32], the formula suggested by Pedro et al. [19], and by using Eqs.(13)–(15). It should be noted that the design proposal of Pedro et al. [19] has been slightly changed to make it applicable for hybrid girders, as shown by Eq. (16). The x-axis shows the analytical resistance calculated using the mentioned formulas. The y-axis shows the numerically computed buckling resistance. It can be observed that the buckling capacities predicted by EN 1993-1-5 [32] exhibit the greatest scattering among the evaluated approaches, suggesting that improvement is needed to enhance accuracy and reduce variability for hybrid girders. Table 12 presents statistical measures for the ratio of numerical resistance to analytical resistance using the mentioned formulas. While all formulas

yield mean values greater than 1.0, indicating generally safe-side predictions, Eqs.(13)–(15) demonstrate the lowest coefficient of variation (CoV) at 0.051, implying greater consistency and reliability in estimating buckling capacity compared to the other methods.

$$c = a \cdot 1,6 \cdot \left(\frac{b_f t_f^2}{h_w^2 t_w} \right)^{0,44} \cdot \frac{f_{yf}}{f_{yw}} \quad (16)$$

Based on the numerical results, it can be concluded that the new formula provides the most reliable estimation of buckling capacity. The proposed approach is founded on a clear analytical framework,

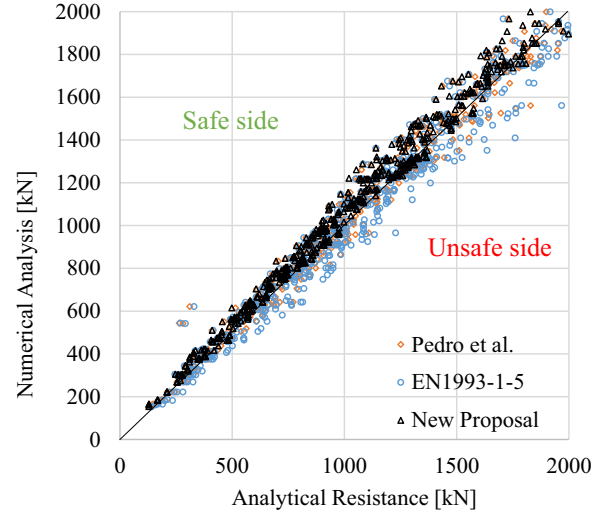


Fig. 18. Comparison between the analytical resistance calculated using several proposals and the numerical resistance.

Table 12

Statistical measures of the ratio of the numerical results to the analytical resistance calculated using the available formulas to estimate the c value.

	By using Eqs. (13)–(15)	By using c of EN 1993-1-5	By using modified c of Pedro et al. [19] acc. to Eq. (16)
Min	0.916	0.777	0.832
Max	1.250	1.243	1.245
σ	0.055	0.079	0.063
μ	1.061	1.001	1.028
CoV	0.051	0.079	0.061

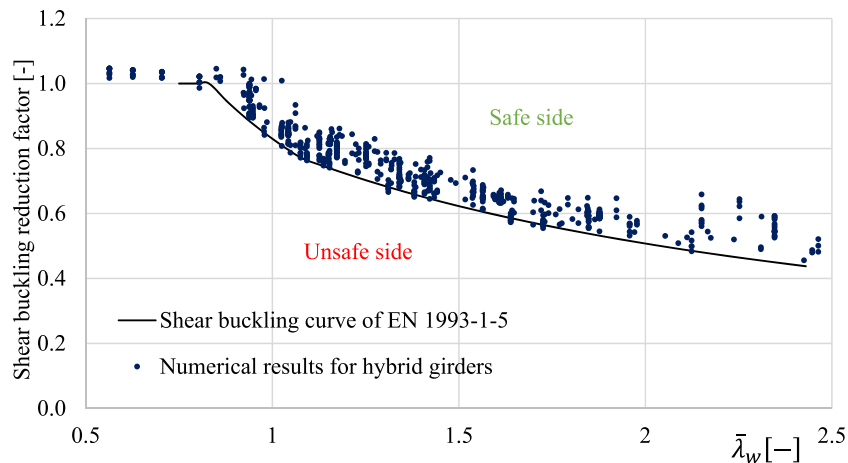


Fig. 17. The shear buckling slenderness vs the shear buckling reduction factor.

Table 13

Reliability assessment of the analyzed combinations of imperfection for hybrid girders.

Formula	V_{fy}	V_{hw}	V_t	b	V_δ	V_{rt}	V_r	γ_{M^*}
acc. to EN1993-1-5	0.07	0.005	0.05	1.003	0.079	0.045	0.009	1.11
acc. to Eqs. (13)–(15)	0.07	0.005	0.05	1.061	0.051	0.045	0.005	0.99

enhancing its credibility and applicability. Its ability to offer consistent and safe predictions, as demonstrated by its lower coefficient of variation, further supports its suitability for practical use in structural design. As shown in Table 12, the proposed formula provides a slightly conservative estimation of the buckling resistance for girders with highly slenderness webs ($\bar{\lambda}_w > 2.2$). Conversely, for girders with lower slenderness ratios ($\bar{\lambda}_w < 0.9$), the formula may slightly overestimate the resistance; however, the maximum observed deviation does not exceed 9 %.

However, it can also be observed, that the modified proposal for hybrid girders of Pedro et al. and the results by using Eqs. (13)–(15) are just slightly different. The final results on the shear buckling resistance is negligible, and both design equations would lead to similar buckling resistances.

5. Statistical evaluation of the results

A statistical analysis is conducted to assess the reliability of the proposed formulas in comparison to GMNIA results. This analysis follows the guidelines outlined in EN1990 Annex D [33]. In this analysis, the obtained value of the buckling resistance using GMNIA is considered the experimental resistance (r_e), while the value obtained using the proposed formulas is considered the theoretical resistance (r_t). According to different studies on the reliability of the shear buckling resistance, the uncertainties of the material and the geometry are considered [34]. Further details of the performed study can be found in a previous publication by the authors [35,36]. The current paper presents only the final results of the analysis. Table 13 summarises the findings, including the utilized formula to calculate c , the uncertainties of the basic variables, including: the yield strength (V_{fy}), the height of the web (V_{hw}), and the thickness of the web (V_t). Additionally, Table 13 reports the mean correction factor (b), the coefficient of variation of the theoretical model (V_δ), the coefficient of variation of the basic variables (V_{rt}), the coefficient of variation of the model (V_r), and the corrected partial safety factor (γ_{M^*}). The corrected partial safety factor is calculated based on nominal values of the basic variables, incorporating the overstrength factor of the steel material to determine γ_{M^*} [35,36]. The presented method is proposed by Afshan et al. [34,37] to utilize the dependency on the basic variables to provide a less conservative estimation of the partial safety factor.

The results reveal that the partial safety factor (γ_{M^*}) derived from the EN 1993-1-5 [11] formula is conservative, yielding the highest value of 1.11. This is close to the value determined by [38], which is equal to 1.07. In contrast, the newly developed formula presented in Eqs. (13)–(15) provides the most favourable method, with a corrected partial safety factor of 0.99. This improvement suggests that the proposed method offers a more efficient and sufficiently safe approach to quantifying the contribution of flanges, thereby enhancing both the accuracy and reliability of the structural assessment. It is important to note that the proposed formula has been validated only within the parametric range presented in Table 9. Therefore, additional verification is required to confirm its applicability beyond the specified limits.

6. Conclusions

The research presented in this study investigated the contribution of flange size to the shear buckling capacity of slender steel plate girders.

First, a detailed numerical model is developed to simulate the behavior of plate girders under shear loading. This model is carefully validated against experimental data to ensure its accuracy in predicting the shear response of the girders. Once validated, the model is used to conduct a comprehensive parametric study, investigating a range of configurations, including both homogeneous and hybrid girders. The parametric study varied key geometric factors such as flange width, flange thickness, and web slenderness to assess their effects on the shear buckling capacity and the distance of the plastic hinge, denoted as c . In particular, the influence of flange stiffness on the value of c and the buckling capacity of the hybrid girders.

The findings from the numerical simulations revealed that for hybrid girders with the same geometrical configurations but with an increased yield strength of the flanges, there is an increase in the distance of the plastic hinge from the end post; however, this increase in c is not directly related to the estimated buckling capacity. However, the available formula in the Eurocode estimates a maximum difference of 10 %. Therefore, an improved formula is developed to estimate the distance c more accurately, accounting for the contribution of flange stiffness and also for hybrid girders. This newly proposed formula offers a better approximation of the shear resistance in slender hybrid girders and provides a more reliable tool for designing shear-critical steel structures. The provided formula has a clear analytical background and does not depend on calibration against numerical tests, leading to a more efficient and optimized solution that can be applied to hybrid graders.

The proposed design equation can be combined with the previous design proposal of Pedro et al. [19], which design equation is found to be the most reliable for non-hybrid girders. The previous design equation is also modified to be applicable for hybrid girders.

CRedit authorship contribution statement

E. Bärnkopf: Writing – original draft, Validation, Software, Investigation. **M. Radwan:** Writing – original draft, Software, Methodology, Investigation, Formal analysis, Data curation. **B. Kövesdi:** Writing – review & editing, Visualization, Supervision, Resources, Methodology, Conceptualization.

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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Data availability

Data will be made available on request.

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