

Behaviour and design of S960 ultra-high strength steel non-slender welded I-section beam–columns

Andi Su^{a,b}, Yuyin Wang^{a,b}, Yajin Wang^a, Kim J.R. Rasmussen^c, Leroy Gardner^{d,*}

^a Key Lab of Structures Dynamic Behavior and Control of the Ministry of Education, Harbin Institute of Technology, Harbin 150090, China

^b Key Lab of Smart Prevention and Mitigation of Civil Engineering Disasters of the Ministry of Industry and Information Technology, Harbin Institute of Technology, Harbin 150090, China

^c School of Civil Engineering, The University of Sydney, Sydney, NSW 2006, Australia

^d Department of Civil and Environmental Engineering, Imperial College London, UK

ARTICLE INFO

Keywords:

Beam–columns
Design analysis
Finite element modelling
Non-slender welded I-sections
S960 UHSS
Structural behaviour

ABSTRACT

The structural behaviour of S960 ultra-high strength steel (UHSS) welded I-section beam–columns is studied in the present paper. Nonlinear finite element (FE) models were first created and validated against relevant test results collected from the literature; the models were then used to carry out parametric studies to obtain additional FE data over an extensive spectrum of cross-section dimensions, member lengths and loading combinations. Given the lack of existing design rules for S960 UHSS structures, the codified beam–column interaction curves prescribed in the current European (EC3) and American (AISC 360) design specifications for lower strength steel members, were assessed for their applicability to the studied S960 UHSS non-slender welded I-section members, utilizing the generated numerical data. It was found that EC3 yielded conservative failure load predictions for Class 1 and 2 S960 UHSS welded I-section beam–columns buckling about either principal axis; similar results were found for Class 3 members under combined compression and strong-axis bending, with the mean FE to predicted failure load ratio being 1.29, the corresponding COV being 0.12, but rather conservative strength predictions arose for Class 3 members under combined compression and weak-axis bending, with the mean FE to predicted failure load ratio being 1.50. AISC 360 led to overall accurate and consistent failure load predictions for S960 UHSS non-slender welded I-section beam–columns, but resulted in some unconservative strength predictions for members under combined compression and weak-axis bending. New design proposals, adopting the format of EC3 design interaction curve, but with more accurate compression and bending end points, interaction factors and buckling reduction factors determined from the continuous strength method (CSM), were proposed and shown to lead to improved design accuracy.

1. Introduction

The utilization of ultra-high strength steel (UHSS) in structural engineering can offer a range of benefits, including lighter and more sustainable construction [1]. However, its current use is limited, to some extent, by the absence of appropriate structural design provisions and fabrication challenges. The structural performance of S960 UHSS members of various cross-section profiles subjected to different loading scenarios has thus been investigated in a number of recent studies, providing a basis for the development of efficient design rules. Regarding the material properties and compressive behaviour, Su et al. [2] conducted thorough measurements of the material properties and membrane residual stresses in S960 UHSS welded I-sections. The

acquired material stress–strain curves were more rounded with little (if any) yield plateau, much higher yield and ultimate stresses, as well as lower ductility, in comparison with normal strength steels (NSS). The measured residual stresses in S960 UHSS welded I-sections had lower normalised (by the yield stress) peak values and slightly shorter transition zones from tensile to compressive residual stresses than those in NSS welded I-sections. S960 UHSS welded I-section stub column tests were undertaken in [2–4], where the compressive strengths and cross-sectional structural responses were carefully studied, verifying the suitability of the effective width formulations and design provisions for 690 MPa (or 700 MPa) high strength steel (HSS) plate elements (i.e. webs and flanges) to their S960 UHSS counterparts. Press-braked channel and angle section stub columns made up of S960 UHSS have

* Corresponding author.

E-mail address: leroy.gardner@imperial.ac.uk (L. Gardner).

<https://doi.org/10.1016/j.engstruct.2024.117602>

Received 26 October 2023; Received in revised form 28 December 2023; Accepted 30 January 2024

Available online 13 February 2024

0141-0296/© 2024 The Author(s). Published by Elsevier Ltd. This is an open access article under the CC BY license (<http://creativecommons.org/licenses/by/4.0/>).

also been experimentally examined by Wang et al. [5], with their compressive behaviour and strengths investigated, as well as the drawbacks of the current codified design methods highlighted. The global stability of welded I- and box-section long columns made of S960 UHSS was examined and presented in [6–9], where the codified buckling curves were found to be rather conservative and new buckling curves, which significantly improve the design consistency and accuracy, were proposed. The weak-axis global buckling behaviour of press-braked channel section columns made from S960 UHSS was also explored by Wang et al. [10].

In terms of bending behaviour, Su et al. [11] carried out extensive laboratory tests and numerical simulations on S960 UHSS welded I-section beams failing by in-plane flexure, and highlighted that the current European, American and Australian standards led to consistent and accurate bending capacity predictions for Class 1 and 2 (or compact) S960 UHSS welded I-sections, but conservative bending capacity predictions for Class 3 (or non-compact) and Class 4 (or slender) S960 UHSS welded I-sections. Experimental and numerical investigations into the structural response of press-braked S960 UHSS channel section beams were conducted and reported in [12], where the current codified rules were revealed to result in safe-sided, but rather scattered and conservative, ultimate moment predictions. Bradford and Liu [13] investigated numerically the lateral-torsional buckling behaviour of S960 UHSS welded I-sections subjected to strong-axis bending; the bending capacities of S960 UHSS welded I-sections were found to be higher, on a normalized basis, than those fabricated from S690 HSS and NSS, owing to the residual stresses being relatively less severe for S960 UHSS welded I-sections; current design methods were revealed to offer rather conservative bending capacity predictions and new design recommendations were made. The lateral-torsional buckling behaviour of S960 UHSS welded I-section beams was also explored numerically by Wang et al. [14]; new design equations were proposed for beams failing by coupled lateral-torsional and local buckling. These studies have highlighted some shortcomings in the current design rules when used for predicting the bending capacities of S960 UHSS components.

For combined loading cases, the cross-sectional behaviour and strengths of eccentrically loaded S960 UHSS slender welded I-section stub columns subjected to weak-axis and strong-axis combined loading were respectively reported in [15] and [16], where the codified interaction curves for compression plus weak-axis bending were found to lead to scattered and overly conservative resistance predictions, principally due to the inaccurate end point for cross-section bending resistance, whereas the codified interaction expressions for compression plus strong-axis bending were shown to lead to conservative but consistent resistance predictions. The structural response and design of S960 UHSS non-slender welded I-sections subjected to combined loading were investigated by Su et al. [17], where existing codified interaction curves were evaluated for their applicability to S960 UHSS and the continuous strength method (CSM) was found to offer improved design consistency and accuracy relative to current codified provisions. Su et al. [18] also systematically studied the structural response of S960 welded I-section beam-columns with slender cross-sections by means of physical testing and numerical modelling; existing interaction curves were again shown to result in rather scattered and conservative strength predictions, and new design proposals, with more accurate bending and compression end points, were put forward. Fang and Chan [19] conducted a numerical study of welded box section beam-columns made up of S960 UHSS plates, in which the effects of residual stresses and varying load eccentricity were investigated; on the basis of the generated numerical data, the current design rules were again revealed to be unduly conservative. To date, the structural behaviour and design of S960 UHSS non-slender welded I-section beam-columns remain unexplored, and are thus the subject to this paper.

The structural performance and design of S960 UHSS non-slender welded I-section beam-columns are investigated in the present paper. Existing test results were first collected. Finite element (FE) models were

then created and validated with reference to the collected data, which comprised results from eight experiments on S960 UHSS welded I-section beam-columns [18]. Upon validation of the FE models, extensive parametric studies were conducted, where 554 FE results on non-slender beam-columns were generated, with 254 bending about the strong principal axis and 300 bending about the weak principal axis, including a wide range of cross-section sizes, member lengths and loading combinations. Given the current lack of design rules for S960 UHSS structures, the applicability of the existing provisions specified in EN 1993-1-12 [20] and AISC 360 [21] was evaluated; the findings are discussed and new design proposals are put forward.

2. Finite element modelling

2.1. General

Finite element (FE) modelling of S960 UHSS beam-columns is carried out in this section. The numerical analyses were performed utilizing the nonlinear FE package ABAQUS [22]. The detailed techniques and assumptions adopted for the establishment of the FE models are first presented. This is followed by a validation study, with the aim of validating the established FE models with reference to the experimental data presented by Su et al. [18]. Finally, the validated FE models are utilized in parametric studies to derive additional results over an extensive spectrum of loading combinations, member lengths and cross-section dimensions.

2.2. Establishment of FE models

The shell element S4R [22] was selected for discretising the I-section beam-column FE models, as widely employed in numerical simulations of metallic members [23–28]. An initial study of the mesh sensitivity was conducted, aimed at selecting a suitable finite element size that enabled the residual stress distribution to be appropriately incorporated and meanwhile led to reasonable computational efficiency and accuracy; finite element sizes varying in the range of $0.4t$ – $6t$ were considered and the final chosen size was equal to the material thickness t . In terms of the material modelling of S960 UHSS, the measured elastic modulus (E), as presented in Table 2, was adopted in the numerical simulations, while the engineering stress–strain response, as acquired from the tensile coupon tests reported in [18], was converted into a true stress–plastic strain response using Eqs. (1) and (2), before being inputted into ABAQUS [22], where σ_{true} and ϵ_{ln}^{pl} are respectively the true stress and plastic strain, and σ_{nom} and ϵ_{nom} are the engineering stress and strain, respectively. The engineering as well as the true plastic stress–strain curves for the two longitudinal coupons tested in [18] (i.e. cut parallelly to the virgin plate rolling direction) are depicted in Fig. 1, while the key material characteristics for these two coupons are summarized in

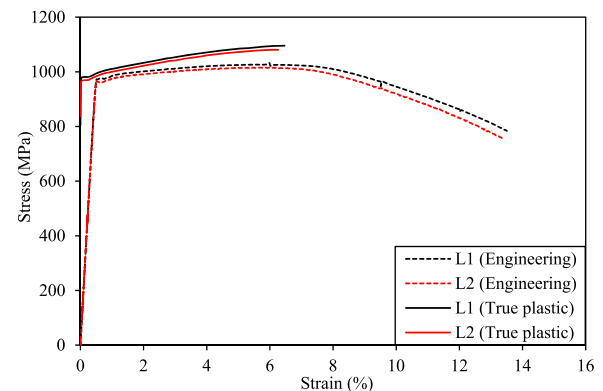


Fig. 1. Engineering and true plastic stress–strain relationships from longitudinal tensile coupons tested in [2].

Table 1

Summary of key measured material properties of S960 UHSS longitudinal coupons.

Longitudinal coupon	E (MPa)	f_y (MPa)	f_u (MPa)	ε_u (%)	ε_f (%)	f_u/f_y
L1	204100	976	1032	6.0	16.0	1.06
L2	204700	962	1016	5.7	16.0	1.06

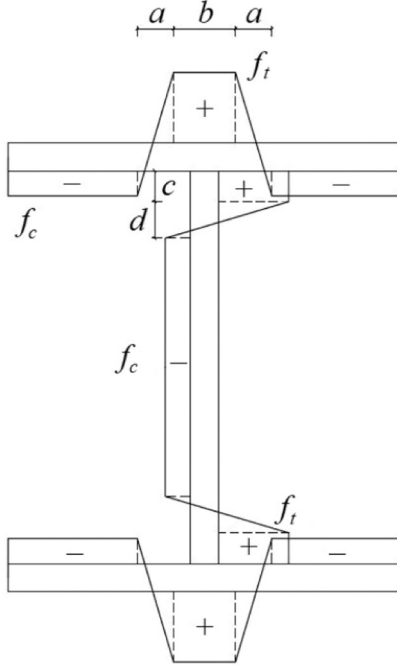


Fig. 2. Predictive model for membrane residual stresses in S960 UHSS welded I-sections [2].

Table 2

Predictive model for residual stresses in S960 UHSS welded I-sections.

Peak tensile residual stress f_t	Peak compressive residual stress f_c	a	b	c	d
$0.8f_y$	From equilibrium	$0.075b_f$	$0.15b_f$	$0.075h_w$	$0.075h_w$

Note: a , b , c and d are the distribution parameters; h_w is the clear distance between the flanges.

Table 1. The average of the two curves was used in the FE models.

$$\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \quad (1)$$

$$\varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{nom}}{E} \quad (2)$$

For each FE model, the membrane residual stresses were firstly determined on the basis of the previously proposed model by Su et al. [2], with the predictive model profile and relevant parameters shown respectively in Fig. 2 and Table 2, and then incorporated into Abaqus utilizing the ‘Predefined Field’ command in ABAQUS [22]. Further information on residual stresses in welded steel I-sections of different grades is provided in [2,28,29]. The boundary conditions of the beam-column FE models were assigned to simulate pinned ends. For each FE model, all nodes in the two end cross-sections of the members were coupled to reference points (RPs). The two RPs were positioned eccentrically with respect to the cross-sectional weak or strong principal axis, and also longitudinally offset from the end sections by 70 mm – the distance between the centre of rotation of the knife-edge assembly and the specimen end reported in the experiments performed by Su et al.

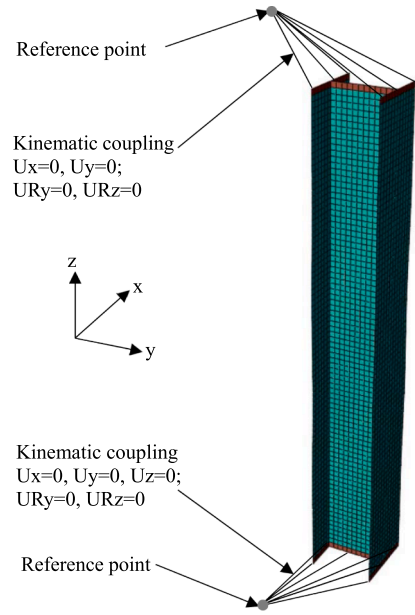


Fig. 3. Coupling details and boundary conditions defined in FE model under combined compression and strong-axis bending.

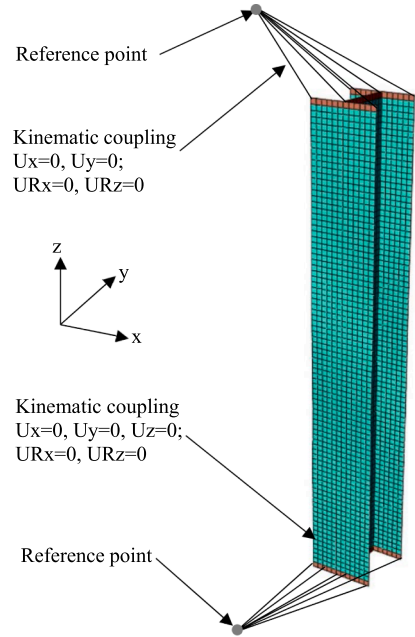


Fig. 4. Coupling details and boundary conditions defined in FE model under combined compression and weak-axis bending.

[14,15,18]. The RPs were restrained against all degrees of freedom except for translation along the member longitudinal axis at one RP and rotation about the axis of combined loading at both RPs; the details of the couplings and defined boundary conditions are shown in Figs. 3 and 4, where U_x , U_y and U_z are translations along X , Y and Z axis, respectively, and UR_x , UR_y and UR_z are rotations about the corresponding axis.

In view of the fact that initial geometric imperfections can have a significant influence on the structural response of slender metallic components, the original perfect geometry of each FE model was perturbed with both local and global imperfections [30–33]. The initial global and local geometric imperfection distribution profiles were taken as the corresponding lowest elastic global and local buckling mode shapes [2,8,9,34,35]. For the validation study, five combinations of

imperfection magnitudes, comprising the measured values of local + global imperfections ($\omega_0 + \omega_g$) reported by Su et al. [18] and four generalized values $-L/1000 + t/100$, $L/1500 + t/100$, $L/1000 + t/10$ and $L/1500 + t/10$, were employed to scale the eigenmodes for the purpose of assessing the effect of the imperfection magnitudes on the ultimate strengths and for selecting the most suitable imperfection combination for further parametric studies. Finally, nonlinear Riks analyses [22] were performed on the imperfect FE models to determine the load–deformation equilibrium paths and failure modes.

2.3. Validation of FE models

Validation of the established FE models was carried out by comparing the numerical results with the experimental observations presented by Su et al. [18]. A brief review of the test results is first provided herein. Two welded I-section geometries were examined – I-120 × 120 × 6 and I-150 × 75 × 6. For each section, four beam–columns were tested about the weak principal axis with different loading eccentricities; each tested specimen was labelled, with the specimen ID comprising a letter 'I' denoting I-section, a number '1' or '2' respectively representing the cross-section geometry I-120 × 120 × 6 and I-150 × 75 × 6 and a letter 'E' representing eccentricity, followed by a number indicating the loading eccentricity in mm. The key geometric dimensions of the test specimens, involving the member length L , the plate thickness t , which was the same for the web and flanges, the flange width b_f , the outer height h (see Fig. 5 for the definitions of the aforementioned cross-sectional geometric parameters), the initial local and global geometric imperfection magnitudes ω_0 and ω_g , and the key beam–column test results, comprising the measured initial loading eccentricity e_0 , failure load N_u , the mid-height lateral deflection at the failure load δ_u , and ultimate moment M_u , are summarized in Table 3. The residual stresses modelled for the representative specimen I1-E99.8 is shown in Fig. 6. For the validation of the FE models, the numerically and experimentally obtained load–lateral deflection histories of all the specimens at mid-height were compared, as depicted in Fig. 7, in which the FE responses are observed to accurately replicate the experimental results. Fig. 8 displays the FE and test failure modes for the representative specimen I1-E60.0, showing excellent agreement. The FE-to-test ultimate load ratios for all specimens are tabulated in Table 4, indicating that the imperfection combination $L/1000 + t/100$ results in the closest match, with the mean FE-to-test ultimate load ratio and corresponding COV being 0.99 and 0.03, respectively. To conclude, the established FE models can accurately simulate the structural behaviour of S960 UHSS non-slender welded I-section beam–columns and are considered to be suitable for performing parametric studies.

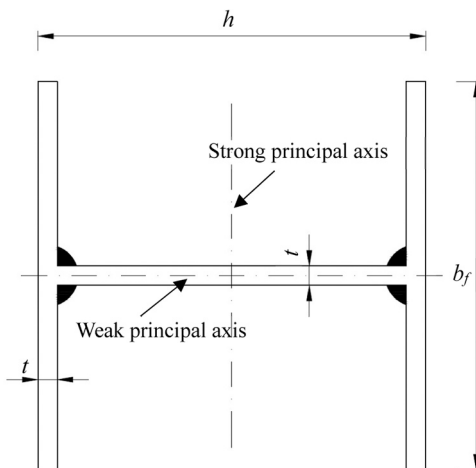


Fig. 5. Notation of S960 UHSS welded I-sections.

2.4. Parametric studies

Parametric studies, performed using the validated FE models, have been carried out to derive additional numerical data over an extensive spectrum of cross-sectional dimensions (and aspect ratios), member lengths and loading combinations; the results are presented in this subsection. Material properties of S960 UHSS, as presented in Fig. 1 and Table 1, were adopted throughout the parametric studies. All the modelled I-sections were non-slender (i.e. Class 1–3), according to the classification limits prescribed in EN 1993–1-12 [20]. Specifically, the overall modelled cross-sectional dimensions (outer height × flange width) were 100 × 100, 120 × 90, 150 × 100, 120 × 70 and 150 × 75, with the flange and web thicknesses varied between 7.0 mm and 13.0 mm, enabling a broad variety of cross-sectional aspect ratios and plate slendernesses to be explored. The modelled lengths of the members were varied between 500 mm and 2500 mm, resulting in a wide range of member non-dimensional slenderness values varying from 0.44 to 3.52. The loading eccentricities were varied between 1 mm and 250 mm, enabling an extensive set of loading combinations to be examined. The magnitudes of the local and global imperfections were respectively assigned as $t/100$ and $L/1000$, which were shown in Table 4 to provide an accurate replication of the experimental responses determined by Su et al. [18]. All the modelled geometric dimensions, as described above, are detailed in Table 5. In total, 554 FE models of S960 UHSS non-slender welded I-section beam–columns were generated from the parametric studies, with 254 bending about the strong principal axis and 300 about the weak principal axis.

3. Assessment of current design rules

3.1. General

Given the absence of existing design rules for S960 UHSS structures, the codified design provisions prescribed in EN 1993–1-12 [20] and AISC 360 [21] for S690 (or S700) HSS non-slender welded I-section beam–columns were assessed for their applicability to their S960 UHSS counterparts in this section, based on comparisons of the numerical failure loads N_u with the unfactored failure loads $N_{u,pred}$ predicted by the considered design codes. The parameter θ , as shown in Fig. 9 and defined by Eq. (3) [36,37], is used to represent different combinations of compression and bending moment, where $M_{u,pred}$ is the predicted failure moment and can be calculated as the product of the predicted failure load $N_{u,pred}$ and the loading eccentricity ($e_0 + \omega_g$); $N_{b,Rd}$ and $M_{b,Rd}$ are the member buckling resistances under compression and bending, respectively. It is worth noting that when θ changes from 0° to 90°, the loading condition varies from pure bending to pure compression.

$$\theta = \tan^{-1} \left(\frac{N_{u,pred}/N_{b,Rd}}{M_{u,pred}/M_{b,Rd}} \right) \quad (3)$$

3.2. EN 1993-1-12 (EC3)

For the design of HSS non-slender welded I-section beam–columns up to grade S700, EN 1993–1-12 [20] employs the same axial load–bending (N–M) interaction curves as specified in EN 1993–1-1 [38], as expressed by:

$$\frac{N_{u,EC3}}{N_{b,y,EC3}} + k_{yy} \frac{M_{u,y,EC3} + \Delta M_{u,y,EC3}}{\chi_{LT} M_{c,y,EC3}} + k_{yz} \frac{M_{u,z,EC3} + \Delta M_{u,z,EC3}}{M_{c,z,EC3}} \leq 1 \quad (4)$$

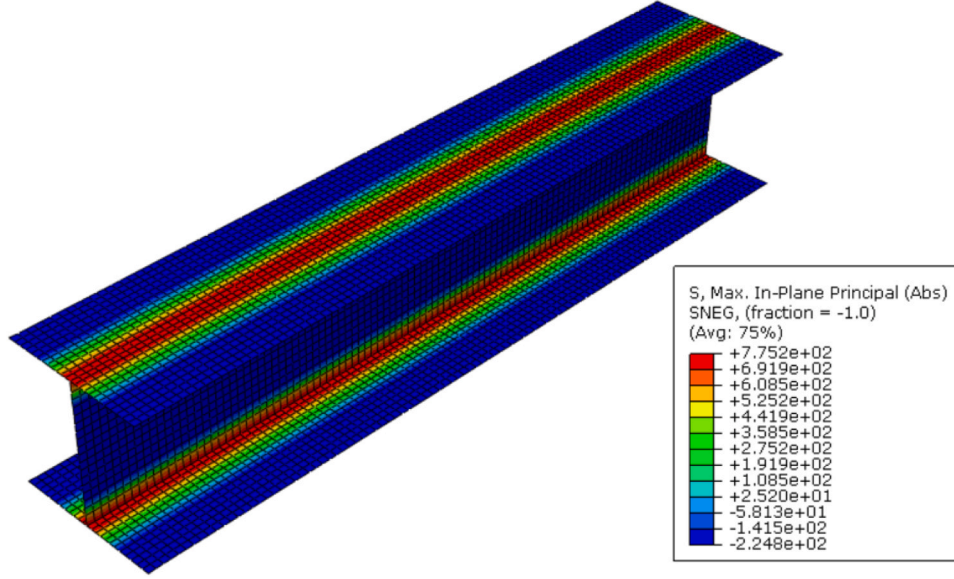
$$\frac{N_{u,EC3}}{N_{b,z,EC3}} + k_{zy} \frac{M_{u,y,EC3} + \Delta M_{u,y,EC3}}{\chi_{LT} M_{c,y,EC3}} + k_{zz} \frac{M_{u,z,EC3} + \Delta M_{u,z,EC3}}{M_{c,z,EC3}} \leq 1 \quad (5)$$

where k_{yy} , k_{zz} , k_{yz} and k_{zy} are four interaction factors; for welded I-section members under combined compression and strong-axis bending, k_{yy} and k_{zy} can be respectively calculated from Eqs. (6) and (7), while under

Table 3

Summary of geometric dimensions and key test results for S960 UHSS slender welded I-section beam–column specimens [18].

Cross section	Specimen ID	L (mm)	h (mm)	b_f (mm)	t (mm)	ω_0 (mm)	ω_g (mm)	e_0 (mm)	N_u (kN)	δ_u (mm)	M_u (kN-m)
I-120 × 120 × 6	I1-E20.0	661	119.36	119.23	5.99	0.07	0.45	20.0	978.7	12.7	32.5
	I1-E31.6	660	119.28	119.39	5.98	0.06	0.43	31.6	789.4	13.7	36.1
	I1-E60.0	662	119.89	119.98	6.04	0.06	0.40	60.0	563.3	15.4	42.7
	I1-E99.8	659	119.64	119.57	5.99	0.09	0.43	99.8	348.7	15.2	40.2
I-150 × 75 × 6	I2-E21.5	551	150.25	74.96	6.01	0.07	0.31	21.5	452.6	15.6	16.9
	I2-E29.4	550	150.20	74.71	6.02	0.09	0.37	29.4	393.1	15.1	17.7
	I2-E40.0	550	150.01	74.59	6.01	0.09	0.30	40.0	299.9	16.2	16.9
	I2-E88.1	551	150.12	74.52	6.03	0.07	0.29	88.1	156.2	18.9	16.7

**Fig. 6.** Typical membrane residual stress pattern and amplitudes (in MPa) in modelled I1-E99.8 specimen.

combined compression and weak-axis bending, k_{zz} and k_{yz} can be respectively obtained from Eqs. (8) and (9), where C_m is the equivalent uniform moment factor, taken as unity in the current study.

$$k_{yz} = \begin{cases} 0.6k_{zz} & \text{for Class 1/2 I – sections} \\ k_{zz} & \text{for Class 3 I – sections} \end{cases} \quad (9)$$

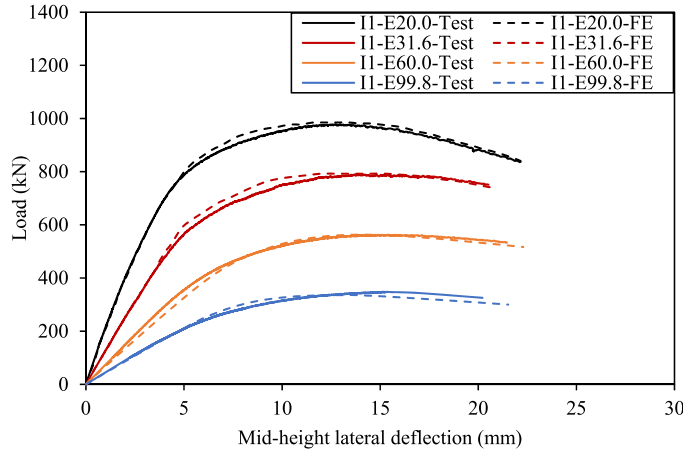
The term $N_{u,EC3}$ is the EC3 predicted failure load, while $M_{u,y,EC3}$ and

$$k_{yy} = \begin{cases} C_m \left(1 + (\bar{\lambda}_y - 0.2) \frac{N_{u,EC3}}{N_{b,y,EC3}} \right) \leq C_m \left(1 + 0.8 \frac{N_{u,EC3}}{N_{b,y,EC3}} \right) & \text{for Class 1/2 I – sections} \\ C_m \left(1 + 0.6\bar{\lambda}_y \frac{N_{u,EC3}}{N_{b,y,EC3}} \right) \leq C_m \left(1 + 0.6 \frac{N_{u,EC3}}{N_{b,y,EC3}} \right) & \text{for Class 3 I – sections} \end{cases} \quad (6)$$

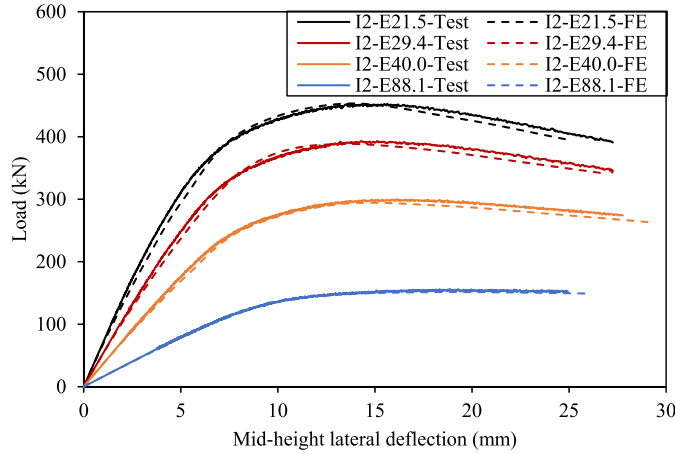
$$k_{zy} = \begin{cases} 0.6k_{yy} & \text{for Class 1/2 I – sections} \\ 0.8k_{yy} & \text{for Class 3 I – sections} \end{cases} \quad (7)$$

$M_{u,z,EC3}$ are respectively the EC3 predicted failure moment about the strong-axis and weak-axis. It is worth noting the loading condition in investigated this paper is combined compression and uniaxial bending, i. e. strong-axis (see Fig. 10(a)) and weak-axis (see Fig. 10(b)) combined loading. In terms of strong-axis combined loading, Eqs. (4) and (5) can

$$k_{zz} = \begin{cases} C_m \left(1 + (2\bar{\lambda}_z - 0.6) \frac{N_{u,EC3}}{N_{b,z,EC3}} \right) \leq C_m \left(1 + 1.4 \frac{N_{u,EC3}}{N_{b,z,EC3}} \right) & \text{for Class 1/2 I – sections} \\ C_m \left(1 + 0.6\bar{\lambda}_z \frac{N_{u,EC3}}{N_{b,z,EC3}} \right) \leq C_m \left(1 + 0.6 \frac{N_{u,EC3}}{N_{b,z,EC3}} \right) & \text{for Class 3 I – sections} \end{cases} \quad (8)$$



(a) For I-120×120×6 specimens.



(b) For I-150×75×6 specimens.

Fig. 7. Comparison of FE and test load–mid-height lateral deflection curves under combined compression and weak-axis bending.



Fig. 8. Comparison of FE and test failure modes for specimen I1-E60.0 [18].

Table 4

Comparison of FE failure loads with test failure loads considering different imperfection magnitudes.

Specimen ID	FE N_u /Test N_u				
	$\omega_g + \omega_0$	$L/1000 + t/100$	$L/1500 + t/100$	$L/1000 + t/10$	$L/1500 + t/10$
I1-E20.0	1.04	1.01	1.02	1.00	1.01
I1-E31.6	0.97	0.94	0.97	0.96	0.96
I1-E60.0	1.02	1.00	0.98	0.97	0.96
I1-E99.8	0.99	0.97	0.94	0.95	0.97
I2-E21.5	1.05	1.03	1.02	1.03	1.04
I2-E29.4	0.99	0.97	0.96	0.94	0.96
I2-E40.0	0.99	0.97	0.99	0.97	0.98
I2-E88.1	1.05	1.02	0.97	0.97	0.99
Mean	1.01	0.99	0.98	0.97	0.98
COV	0.03	0.03	0.03	0.03	0.03

be respectively simplified to Eqs. (10) and (11), since, in this case, $M_{u,z,EC3}$ is equal to zero and $\Delta M_{u,y,EC3}$ and $\Delta M_{u,z,EC3}$, which are the moments due to the shift of the centroidal axes for Class 4 sections, are also equal to zero. Similarly, for weak-axis combined loading, Eqs. (4) and (5) can be simplified to Eqs. (12) and (13), from which, it is clear that Eq. (13) is critical.

$$\frac{N_{u,EC3}}{N_{b,y,EC3}} + k_{yy} \frac{M_{u,y,EC3}}{\chi_{LT} M_{c,y,EC3}} \leq 1 \quad (10)$$

$$\frac{N_{u,EC3}}{N_{b,z,EC3}} + k_{zy} \frac{M_{u,y,EC3}}{\chi_{LT} M_{c,y,EC3}} \leq 1 \quad (11)$$

$$\frac{N_{u,EC3}}{N_{b,y,EC3}} + k_{yz} \frac{M_{u,z,EC3}}{M_{c,z,EC3}} \leq 1 \quad (12)$$

$$\frac{N_{u,EC3}}{N_{b,z,EC3}} + k_{zz} \frac{M_{u,z,EC3}}{M_{c,z,EC3}} \leq 1 \quad (13)$$

where $N_{b,y,EC3} = \chi_y A f_y$ and $N_{b,z,EC3} = \chi_z A f_y$ are the EC3 column buckling resistances about strong and weak principal axes, respectively, in which the buckling reduction factor χ (i.e. the umbrella term for χ_y and χ_z) is given by:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1.0 \quad (14)$$

where $\Phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2]$ and $\bar{\lambda} = \sqrt{A f_y / N_{cr}}$ is the column non-dimensional slenderness with N_{cr} being the elastic critical force for the relevant buckling mode. The parameter α is an imperfection factor; for welded I-sections with flange thicknesses less than 40 mm buckling about the strong-axis, α is equal to 0.34 (i.e. buckling curve b), while for weak-axis buckling, α is equal to 0.49 (i.e. buckling curve c).

The cross-section bending resistance $M_{c,y,EC3}$ (or $M_{c,z,EC3}$) is equal to $W_{pl,y} f_y$ (or $W_{pl,z} f_y$) for Class 1 and 2 welded I-sections and $W_{el,y} f_y$ (or $W_{el,z} f_y$) for Class 3 welded I-sections, where $W_{pl,y}$ (or $W_{pl,z}$) and $W_{el,y}$ (or $W_{el,z}$) are plastic section modulus and elastic section modulus, respectively. The reduction factor for lateral torsional buckling χ_{LT} is given by:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \text{ but } \chi_{LT} \leq 1.0 \quad (15)$$

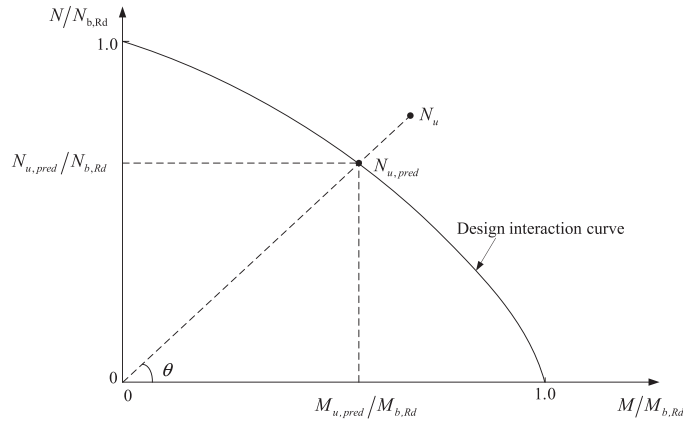
where $\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2]$, in which α_{LT} is an imperfection factor equal to 0.49 for welded I-sections with aspect ratios no greater than 2; $\bar{\lambda}_{LT}$ is the lateral-torsional buckling slenderness, given by:

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{xy} f_y}{M_{cr,y}}} \quad (16)$$

Table 5

Cross-section geometric dimensions selected for parametric studies.

Bending axis	Cross-section dimensions			Aspect ratio	Member length (mm)	Loading eccentricity (mm)
	$h \times b_f$ (mm)	t_w (mm)	t_f (mm)			
Strong axis	100 × 100	7.0–13.0	7.0–13.0	1.0–2.0	500–1500	1–230
	120 × 90					
	150 × 100					
	120 × 70					
	150 × 75					
Weak axis	100 × 100	7.0–13.0	7.0–13.0	1.0–2.0	500–2500	1–250
	120 × 90					
	150 × 100					
	120 × 70					
	150 × 75					

**Fig. 9.** Definition of θ .

where $M_{cr,y}$ is the elastic critical moment for lateral-torsional buckling, W_y is equal to W_{ply} for Class 1 and 2 welded I-sections and W_{ely} for Class 3 welded I-sections. It is worth noting that the reduction factor χ_{LT} need not to be considered (i.e. $\chi_{LT}=1$) for welded I-section members under weak-axis bending.

The applicability of the EC3 design N–M interaction curve to S960 ultra-high strength steel non-slender welded I-section beam–columns is now assessed. Fig. 11 shows the results of the comparisons between the FE failure loads and the EC3 failure load predictions for S960 UHSS non-slender welded I-section beam–columns under combined compression and strong-axis bending. The results reveal rather conservative resistance predictions, particularly for the bending dominated cases, with the mean FE to predicted failure load ratio equal to 1.29 and the corresponding COV of 0.10, as reported in Table 6. Under combined compression and weak-axis bending, rather conservative resistance predictions are again identified for Class 1 and 2 section members, as shown in Fig. 12, but excessively conservative predictions arise for Class 3 section members; this is reflected in Table 6, where the mean FE to predicted failure load ratio is 1.27 for Class 1 and 2 section members under combined compression and weak-axis bending, but 1.50 for Class 3 sections. The particularly conservative results for the Class 3 sections arise towards the bending end point, where the partial spread of plasticity is not captured by the use of the elastic moment capacity [11]. This will be resolved, to some extent, in the upcoming revision to Eurocode 3 through the introduction of the elastic-plastic modulus [39,40].

3.3. AISC 360 (AISC)

A bi-linear N–M interaction curve, as expressed by Eq. (17), is specified in AISC 360 [21] for the design of HSS non-slender welded I-section beam–columns with material yield stresses no greater than 690 MPa:

$$\begin{aligned} \frac{N_{u,AISC}}{N_{b,AISC}} + \frac{8}{9} \delta_{AISC} \frac{M_{u,AISC}}{M_{b,AISC}} &\leq 1 \quad \text{for } \frac{N_{u,AISC}}{N_{b,AISC}} \geq 0.2 \\ \frac{N_{u,AISC}}{2N_{b,AISC}} + \delta_{AISC} \frac{M_{u,AISC}}{M_{b,AISC}} &\leq 1 \quad \text{for } \frac{N_{u,AISC}}{N_{b,AISC}} < 0.2 \end{aligned} \quad (17)$$

where $N_{u,AISC}$ is the AISC predicted failure load, $M_{u,AISC}$ is the AISC predicted failure moment, $\delta_{AISC}=1/(1-N_{u,AISC}/N_{cr})$ is an amplification factor to allow for member second-order effects, in which N_{cr} is the elastic critical buckling force, and $N_{b,AISC}$ is the column compression resistance, expressed by:

$$N_{b,AISC} = \begin{cases} \left(0.658^{\frac{f_y}{f_e}}\right) A f_y & \text{for } \frac{f_y}{f_e} \leq 2.25 \\ 0.877 A f_e & \text{for } \frac{f_y}{f_e} > 2.25 \end{cases} \quad (18)$$

where $f_e = N_{cr}/A$ for members failing by flexural buckling, with N_{cr} determined based on the relevant principal axis.

Under strong-axis bending, the member bending moment resistance $M_{b,AISC}$ can be computed from Eq. (19) for welded I-sections with compact or non-compact flanges and compact webs, from Eq. (20) for welded I-sections with compact flanges and non-compact webs and from Eq. (21) for welded I-sections with non-compact flanges and non-compact webs, respectively.

$$M_{b,AISC} = M_{pl} - (M_{pl} - 0.7M_{el}) \left(\frac{L - L_p}{L_r - L_p} \right) \leq M_{pl} \quad (19)$$

$$M_{b,AISC} = M_{el} - (M_{el} - 0.7M_{el}) \left(\frac{L - L_p}{L_r - L_p} \right) \leq M_{el} \quad (20)$$

$$M_{b,AISC} = M_{el} - (M_{el} - 0.7M_{el}) \left(\frac{b_f/2t - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \quad (21)$$

where L_p is the limiting laterally unbraced length for the limit state of yielding, L_r is the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, and λ_{pf} and λ_{rf} are respectively the compact and non-compact slenderness limits for outstand flanges in compression.

Regarding beam–columns under combined compression and weak-axis bending, the member bending moment resistance $M_{b,AISC}$ is given by Eq. (22) for welded I-sections with compact flanges and compact/non-compact webs and by Eq. (23) for welded I-sections with non-compact flanges and compact/non-compact webs.

$$M_{b,AISC} = M_{pl} \leq 1.6M_{el} \quad (22)$$

$$M_{b,AISC} = M_{pl} - (M_{pl} - 0.7M_{el}) \left(\frac{b_f/2t - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \quad (23)$$

The applicability of the AISC design N–M interaction curve to S960 UHSS non-slender welded I-section beam–columns is assessed through

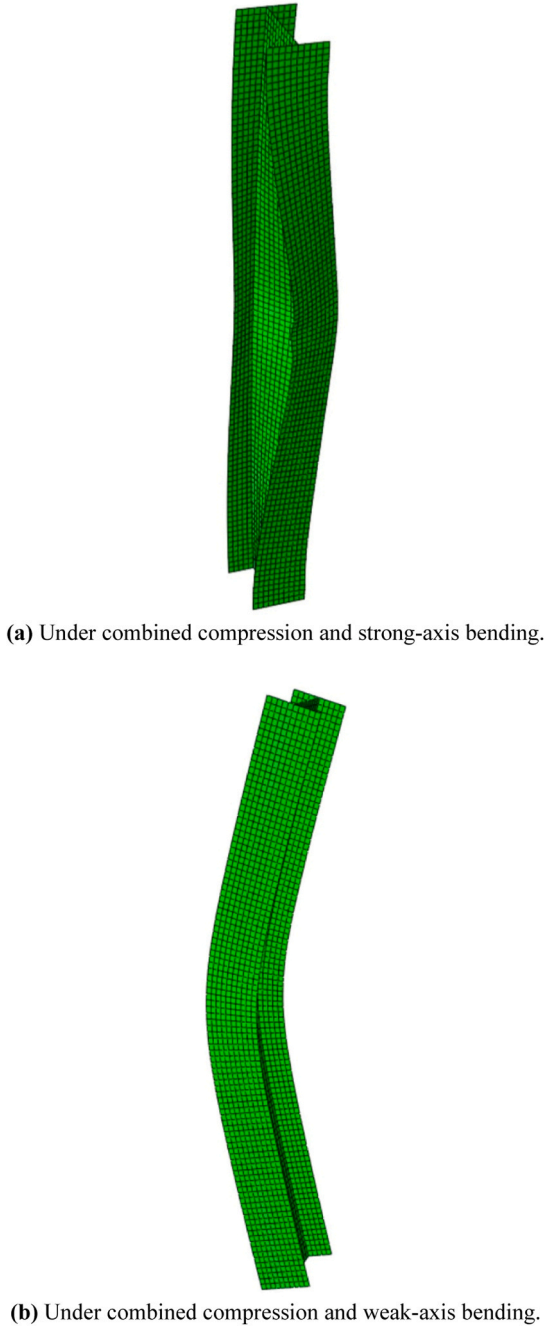


Fig. 10. Failure modes for S960 UHSS non-slender welded I-section beam-columns.

comparisons of the AISC failure load predictions and FE failure loads. Figs. 13 and 14 respectively display the graphical assessment results for members under strong-axis combined loading and weak-axis combined loading, revealing that AISC 360 [21] can provide overall accurate and consistent strength predictions for S960 UHSS non-slender welded I-section beam-columns, as evidenced in Table 7, but with some results on the unsafe side when the loading condition approaches pure bending (i.e. $\theta = 0$) due to the over-allowance for the spread of plasticity (see Eqs. (22) and (23)) in non-compact sections.

4. New design proposals

To address the limitations of the current EC3 and AISC design provisions and to explore viable alternatives, new design proposals, based

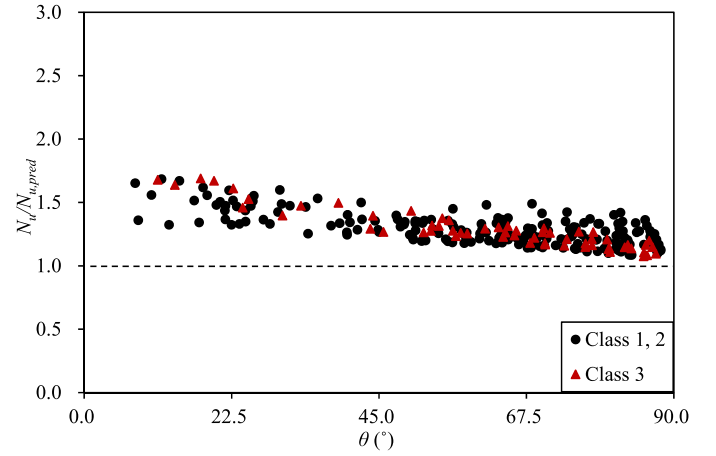


Fig. 11. Comparison of FE failure loads with EC3 failure load predictions for S960 UHSS welded I-section beam-columns under combined compression and strong-axis bending.

Table 6

Comparison of FE failure loads against EC3 failure load predictions.

Bending axis	Class of cross-section	Number of numerical data	$N_u/N_{u,EC3}$	
			Mean	COV
Strong axis	Class 1 and Class 2 I-sections	194	1.29	0.10
	Class 3 I-section	60	1.29	0.12
	Total	254	1.29	0.10
Weak axis	Class 1 and Class 2 I-sections	240	1.27	0.08
	Class 3 I-section	60	1.50	0.06
	Total	300	1.31	0.10

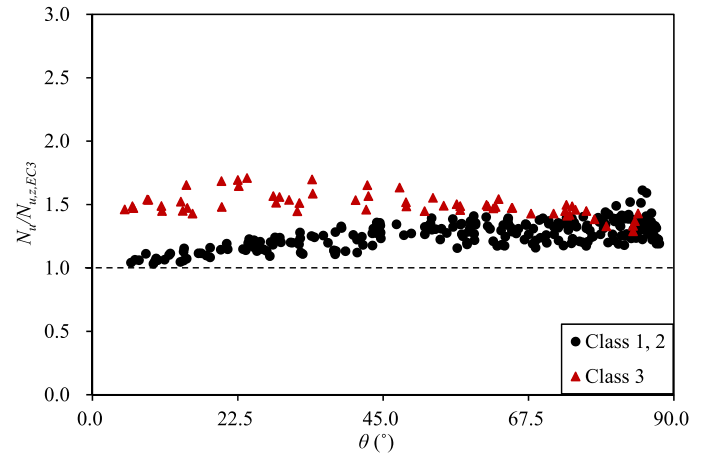


Fig. 12. Comparison of FE failure loads with EC3 failure load predictions for S960 UHSS welded I-section beam-columns under combined compression and weak-axis bending.

on the Continuous Strength Method (CSM) [41–43] for S960 UHSS non-slender welded I-section beam-columns, are put forward in this section. The proposed design interaction curves still follow the same format as used in EC3, but with revised end points, interaction factors and buckling reduction factors, as shown in Eqs. (24)–(26):

$$\frac{N_{u,csm}}{N_{b,y,csm}} + k_{yy,csm} \frac{M_{u,y,csm}}{\chi_{LT,csm} M_{c,y,csm}} \leq 1 \quad (24)$$

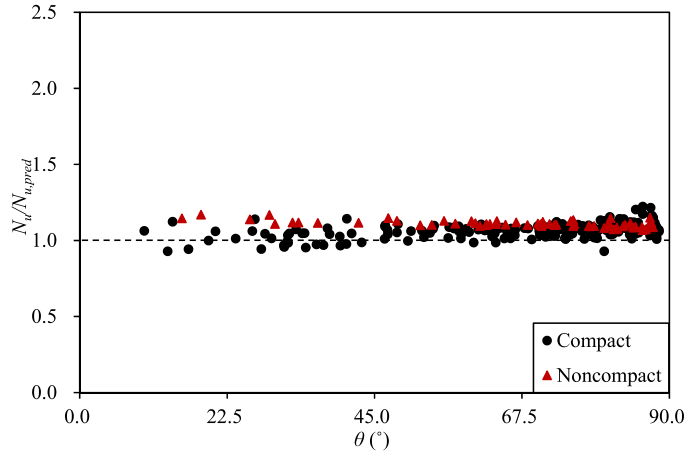


Fig. 13. Comparison of FE failure loads with AISC failure load predictions for S960 UHSS welded I-section beam-columns under combined compression and strong-axis bending.

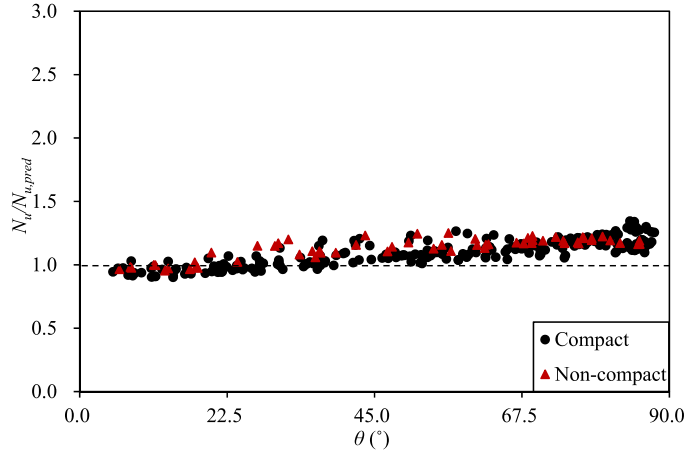


Fig. 14. Comparison of FE failure loads with AISC failure load predictions for S960 UHSS welded I-section beam-columns under combined compression and weak-axis bending.

Table 7

Comparison of FE failure loads against AISC failure load predictions.

Bending axis	Class of cross-section	Number of numerical data	N _u /N _{u, AISC}	
			Mean	COV
Strong axis	Compact I-section	194	1.06	0.05
	Non-compact I-section	60	1.11	0.02
	Total	254	1.07	0.04
Weak axis	Compact I-section	240	1.10	0.09
	Non-compact I-section	60	1.13	0.07
	Total	300	1.11	0.09

$$\frac{N_{u, csm}}{N_{b, z, csm}} + k_{zy, csm} \frac{M_{u, y, csm}}{\chi_{LT, csm} M_{c, y, csm}} \leq 1 \quad (25)$$

$$\frac{N_{u, csm}}{N_{b, z, csm}} + k_{zz, csm} \frac{M_{u, z, csm}}{M_{c, z, csm}} \leq 1 \quad (26)$$

where $N_{u, csm}$ and $M_{u, csm}$ (i.e. the umbrella term for $M_{u, y, csm}$ and $M_{u, z, csm}$) are respectively the CSM-based predicted failure load and moment, and $N_{b, y, csm} = \chi_{y, csm} N_{csm}$ and $N_{b, z, csm} = \chi_{z, csm} N_{csm}$ are respectively the revised column buckling resistances about the strong and weak principal axes,

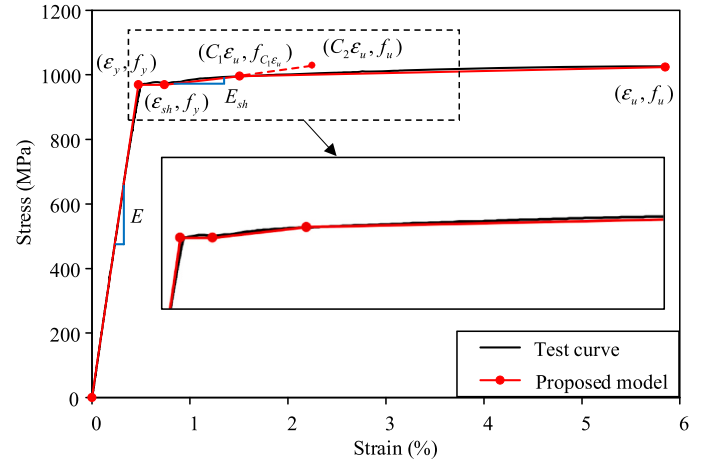


Fig. 15. The proposed quad-linear material model for S960 UHSS [17].

based on CSM, with χ_{csm} determined as described below. Note that Eq. (13) is not critical for the studied cases and thus the equivalent CSM-based curve is not presented. The determination of the CSM cross-section compression capacity N_{csm} requires quantification of the cross-section deformation capacity, which can be determined from the CSM base curve:

$$\frac{\epsilon_{csm}}{\epsilon_y} = \frac{0.25}{\bar{\lambda}_p^{3.6}} \leq \min\left(15, \frac{C_1 \epsilon_u}{\epsilon_y}\right) \text{ for } \bar{\lambda}_p < 0.68 \quad (27)$$

where ϵ_{csm} is the limiting compressive strain for a non-slender welded I-section (i.e. $\bar{\lambda}_p \leq 0.68$) [44], $\epsilon_y = f_y/E$ is the yield strain, ϵ_u is the ultimate strain at the ultimate strength f_u , C_1 is a material coefficient in the adopted quad-linear material model, defining a cut-off strain to avoid over-predictions of material strength, and $\bar{\lambda}_p = \sqrt{f_y/\sigma_{cr}}$ is the cross-section slenderness, in which σ_{cr} is the elastic load buckling stress of the full cross-section and can be calculated using the formulations derived by Gardner et al. [45]. Upon quantification of the limiting compressive strain, a quad-linear material model for S960 UHSS is used for determining the corresponding stresses. Further information in terms of the utilization of the CSM for UHSS structures can be found in Su et al. [17]. The quad-linear material model is depicted in Fig. 15, where C_1 and C_2 are two material coefficients and the strain hardening slope E_{sh} is defined by the material coefficient C_2 , as expressed by Eqs. (21)–(23); the strain hardening strain ϵ_{sh} and the ultimate strain ϵ_u were taken as the average measured values from Fig. 2 and Table 2, being 0.74% and 5.85%, respectively. Fig. 15 shows good agreement between the measured engineering stress–strain curve and the quad-linear material model. Subsequently, the CSM design stress σ_{csm} can be computed using Eq. (24), with the CSM cross-section compression resistance $N_{csm} = A\sigma_{csm}$ also obtained.

$$E_{sh} = \frac{f_u - f_y}{C_2 \epsilon_u - \epsilon_{sh}} \quad (28)$$

$$C_1 = \frac{\epsilon_{sh} + 0.15(\epsilon_u - \epsilon_{sh})}{\epsilon_u} \quad (29)$$

$$C_2 = \frac{\epsilon_{sh} + 0.6(\epsilon_u - \epsilon_{sh})}{\epsilon_u} \quad (30)$$

$$\sigma = \begin{cases} E\epsilon & \text{for } \epsilon \leq \epsilon_y \\ f_y & \text{for } \epsilon_y < \epsilon \leq \epsilon_{sh} \\ f_y + E_{sh}(\epsilon - \epsilon_{sh}) & \text{for } \epsilon_{sh} < \epsilon \leq C_1 \epsilon_u \\ f_{C_1 \epsilon_u} + \frac{f_u - f_{C_1 \epsilon_u}}{\epsilon_u - C_1 \epsilon_u}(\epsilon - C_1 \epsilon_u) & \text{for } C_1 \epsilon_u < \epsilon \leq \epsilon_u \end{cases} \quad (31)$$

The CSM cross-section bending resistance M_{csm} can be obtained from Eqs. (32) and (33), based on the formulations developed by Yun et al. [44], where α and β are two dimensionless coefficients and are respectively set as 1.2 and 0.05 for welded I-sections under combined compression and strong-axis bending, 2.0 and 0.08 for welded I-sections under combined compression and weak-axis bending.

$$M_{csm} = W_{pl} f_y \left[1 - \left(1 - \frac{W_{el}}{W_{pl}} \right) / \left(\frac{\epsilon_{csm}}{\epsilon_y} \right)^\alpha \right] \quad \text{for } \epsilon_{csm} \leq \epsilon_{sh} \quad (32)$$

$$M_{csm} = W_{pl} f_y \left[1 - \left(1 - \frac{W_{el}}{W_{pl}} \right) / \left(\frac{\epsilon_{csm}}{\epsilon_y} \right)^\alpha + \beta \frac{E_{sh}}{E} \left(\frac{\epsilon_{csm} - \epsilon_{sh}}{\epsilon_y} \right)^2 \right] \quad \text{for } \epsilon_{csm} > \epsilon_{sh} \quad (33)$$

The CSM flexural buckling reduction factor χ_{csm} can be obtained from Eqs. (34)–(36):

$$\chi_{csm} = \frac{1}{\Phi_{csm} + \sqrt{\Phi_{csm}^2 - \bar{\lambda}_{csm}^2}} \quad \text{but } \chi_{csm} \leq 1.0 \quad (34)$$

$$\Phi_{csm} = 0.5 [1 + \alpha_{csm} (\bar{\lambda}_{csm} - 0.2) + \bar{\lambda}_{csm}^2] \quad (35)$$

$$\bar{\lambda}_{csm} = \sqrt{N_{csm} / N_{cr}} \quad (36)$$

where the CSM imperfection factor α_{csm} , as proposed by Arrayago et al. [46], can be determined from Eq. (37), in which the EC3 imperfection factor α is respectively equal to 0.34 and 0.49 for strong-axis and weak-axis combined loading and the two coefficients C_3 and C_4 can be derived from Eqs. (38) and (39).

$$\alpha_{csm} = \alpha (C_3 - C_4 \bar{\lambda}_p) \sqrt{\frac{f_y}{\sigma_{csm}}} \frac{N_{csm} M_{el}}{M_{csm} A f_y} \quad (37)$$

$$C_3 = 1 + 0.68 C_4 \quad (38)$$

$$C_4 = 1.2 (f_u / f_y) \quad (39)$$

The CSM lateral-torsional buckling reduction factor $\chi_{LT,csm}$ can be calculated using Eqs. (40)–(43), where the proposed value for the imperfection factor α_{LT} is 0.13 for S960 UHSS non-slender welded I-section members under combined compression and strong-axis bending.

$$\chi_{LT,csm} = \frac{1}{\Phi_{LT,csm} + \sqrt{\Phi_{LT,csm}^2 - \bar{\lambda}_{LT,csm}^2}} \quad \text{but } \chi_{LT,csm} \leq 1.0 \quad (40)$$

$$\Phi_{LT,csm} = 0.5 [1 + \alpha_{LT,csm} (\bar{\lambda}_{LT,csm} - 0.2) + \bar{\lambda}_{LT,csm}^2] \quad (41)$$

$$\bar{\lambda}_{LT,csm} = \sqrt{M_{csm} / M_{cr}} \quad (42)$$

$$\alpha_{LT,csm} = \alpha_{LT} (C_3 - C_4 \bar{\lambda}_p) \sqrt{\frac{f_y}{\sigma_{csm}}} \frac{N_{csm} M_{el}}{M_{csm} A f_y} \quad (43)$$

Given the new CSM end points to the interaction curves, new interaction factors (i.e. $k_{yy,csm}$, $k_{zy,csm}$ and $k_{zz,csm}$) [47] are needed. These are defined in general terms in Eqs. (44–46), with $D_{1,csm} = \gamma D_1$, $D_{2,csm} = D_2 / \gamma$, $D_{3,csm} = D_3 / \gamma$, where $\gamma = \sqrt{f_y / \sigma_{csm}}$ and the values of D_1 , D_2 and D_3 are 1.0, 0.2 and 1.0 for strong-axis combined loading, and 2.0, 0.6 and 1.0 for weak-axis combined loading, as given in EN 1993-1-1 [38].

$$k_{yy,csm} = \begin{cases} C_m \left(1 + (D_{1,y,csm} \bar{\lambda}_{y,csm} - D_{2,y,csm}) \frac{N_{u,y,csm}}{N_{b,y,csm}} \right) & \text{for } \bar{\lambda}_{y,csm} < D_{3,y,csm} \\ C_m \left(1 + (D_{1,y,csm} D_{3,y,csm} - D_{2,y,csm}) \frac{N_{u,y,csm}}{N_{b,y,csm}} \right) & \text{for } \bar{\lambda}_{y,csm} \geq D_{3,y,csm} \end{cases} \quad (44)$$

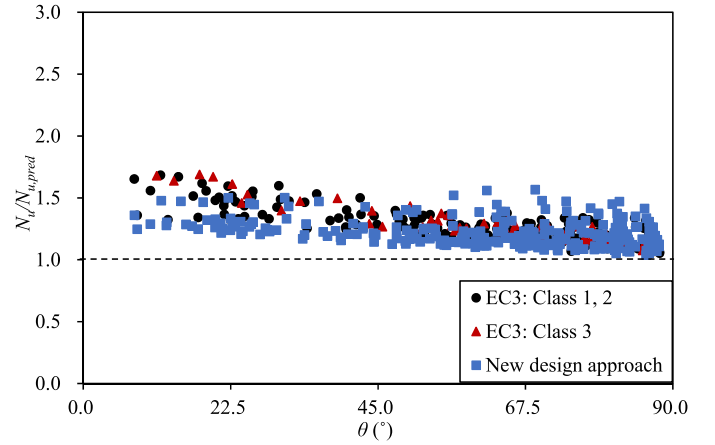


Fig. 16. Comparison of FE failure loads with the predicted failure loads from new design proposals for S960 UHSS welded I-section beam-columns under combined compression and strong-axis bending.

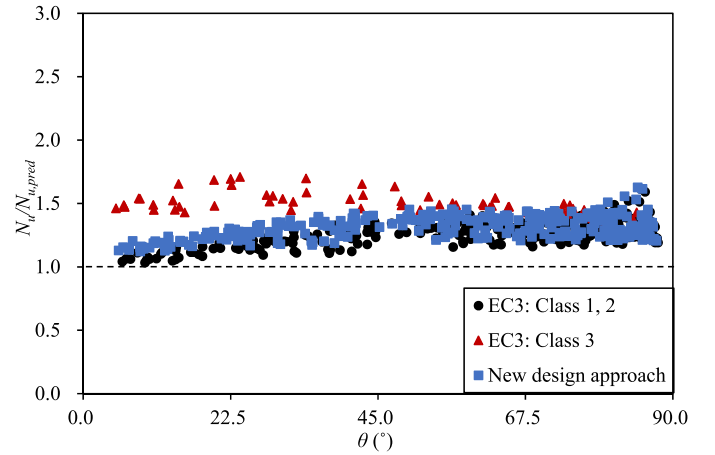


Fig. 17. Comparison of FE failure loads with the predicted failure loads from new design proposals for S960 UHSS welded I-section beam-columns under combined compression and weak-axis bending.

Table 8

Comparison of FE failure loads against the failure load predictions from new CSM-based design proposal.

Bending axis	Class of cross-section	Number of numerical data	$N_u / N_{u,csm}$	
			Mean	COV
Strong axis	Non-slender	254	1.22	0.09
Weak axis	Non-slender	300	1.31	0.07

$$k_{zy,csm} = 0.6 k_{yy,csm} \quad (45)$$

$$k_{zz,csm} = \begin{cases} C_m \left(1 + (D_{1,z,csm} \bar{\lambda}_{z,csm} - D_{2,z,csm}) \frac{N_{u,z,csm}}{N_{b,z,csm}} \right) & \text{for } \bar{\lambda}_{z,csm} < D_{3,z,csm} \\ C_m \left(1 + (D_{1,z,csm} D_{3,z,csm} - D_{2,z,csm}) \frac{N_{u,z,csm}}{N_{b,z,csm}} \right) & \text{for } \bar{\lambda}_{z,csm} \geq D_{3,z,csm} \end{cases} \quad (46)$$

The predicted failure loads determined using the new design proposals are compared with the numerical data points in Figs. 16 and 17, where more accurate predictions from the new proposal compared with EC3 are observed, especially for the Class 3 sections. The results are also reported in Table 8, where the mean $N_u / N_{u,csm}$ ratio and the corresponding COV are seen to be respectively equal to 1.22 and 0.09 for

Table 9

Summary of reliability analysis results for new CSM-based design proposal.

Bending axis	Class of cross-section	Number of numerical data	$k_{d,n}$	b	V_δ	V_r	γ_{M0}^*
Strong axis	Non-slender	254	3.129	1.188	0.089	0.106	1.037
Weak axis	Non-slender	300	3.123	1.361	0.068	0.090	0.858

strong-axis combined loading, and equal to 1.31 and 0.07 for weak-axis combined loading, indicating the improved design accuracy over EC3. The proposed CSM-based design approach is less accurate overall than AISC 360, but AISC 360 yields a number of strength predictions on the unsafe side. The lower accuracy of the CSM-based approach stems primarily from the desire to retain the EC3 interaction equation. Nonetheless, both consistent and safe-sided results are obtained.

5. Reliability assessment of new design proposals

The reliability of the new design proposals for S960 UHSS non-slender welded I-section beam-columns has been assessed through statistical analyses in accordance with Annex D of EN 1990 [48]. In the present reliability analysis, the mean to nominal yield strength ratio f_y , $mean/f_{y,nom}$ (i.e. the material overstrength ratio) for high strength steel is taken equal to 1.135, with a COV of 0.055 [49], the COV of the geometric dimension is taken equal to 0.02 [50]. Table 9 reports the key statistical parameters for the new design approach, where $k_{d,n}$ is the design fractile factor depending on the number of numerical data, b is the correction factor, V_δ is the COV of the numerical resistances relative to the resistance model, V_r is the combined COV incorporating both model and basic variable uncertainties, and γ_{M1} is the partial safety factor. The resulting required partial safety factors γ_{M1} for the design approach, as reported in Table 9, are lower than or very close to the recommended value of 1.00 [20], indicating that the reliability of the proposed design proposals for S960 UHSS non-slender welded I-section beam-columns is acceptable.

6. Conclusions

Extensive numerical simulations have been performed in order to study the structural behaviour of S960 UHSS non-slender welded I-section beam-columns and presented in this paper. Existing test results on S960 UHSS welded I-section beam-columns were first collected. FE models were then established and validated against the test results, including the ultimate loads, failure modes and load-mid-height deflection histories. The validated FE models were afterwards adopted to perform parametric studies to derive additional FE data. The numerical results acquired from the parametric studies were utilized to assess the suitability of current design rules, as prescribed in the European code and American specification, for S960 UHSS non-slender welded I-section beam-columns. The assessment results revealed that (i) the European code results in reasonable failure load predictions for S960 UHSS welded I-section beam-columns, with the mean FE to predicted failure load ratios being 1.29 and 1.31 for the strong- and weak-axis combined loading cases, respectively, indicating the scope for improvement, and (ii) American specification leads to overall accurate and consistent failure load predictions for S960 UHSS non-slender welded I-section beam-columns, but resulted in some unconservative data points for weak-axis combined loading case. A new design approach, adopting the basic format of the EC3 design interaction curve but with more accurate end points, interaction factors and buckling reduction factors determined based on the CSM, were finally proposed and shown to give more accurate and consistent failure load predictions than the current EC3 design provisions.

CRedit authorship contribution statement

Su Andi: Writing – original draft, Visualization, Resources,

Methodology, Investigation, Funding acquisition, Formal analysis, Conceptualization. **Wang Yuyin:** Visualization, Methodology, Investigation, Formal analysis. **Wang Yajin:** Visualization, Methodology, Investigation, Formal analysis. **Rasmussen Kim John Rise:** Writing – review & editing, Supervision, Methodology, Investigation, Formal analysis, Conceptualization. **Gardner Leroy:** Writing – review & editing, Supervision, Methodology, Investigation, Formal analysis, 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.

Data availability

Data will be made available on request.

Acknowledgements

The research carried out and described in this paper was supported by the grant from the National Natural Science Foundation of China (No. 52308163).

References

- [1] IABSE, Use and Application of High-performance Steels for Steel Structures, IABSE, ETH Zürich, Switzerland, 2005.
- [2] Su A, Sun Y, Liang Y, Zhao O. Membrane residual stresses and local buckling of S960 ultra-high strength steel welded I-section stub columns. *Thin Walled Struct* 2021;161:107497.
- [3] Shi G, Zhou W, Lin C. Experimental investigation on the local buckling behavior of 960 MPa high strength steel welded section stub columns. *Adv Struct Eng* 2015;18:423–37.
- [4] Li D, Huang Z, Uy B, Thai H, Hou C. Slenderness limits for fabricated S960 ultra-high-strength steel and composite columns. *J Constr Steel Res* 2019;159:109–21.
- [5] Wang F, Zhao O, Young B. Testing and numerical modelling of S960 ultra-high strength steel angle and channel section stub columns. *Eng Struct* 2020;204:109902.
- [6] Shi G, Ban H, Bijlaard FSK. Tests and numerical study of ultra-high strength steel columns with end restraints. *J Constr Steel Res* 2012;70:236–47.
- [7] Ban H, Shi G, Shi Y, Bradford MA. Experimental investigation of the overall buckling behaviour of 960 MPa high strength steel columns. *J Constr Steel Res* 2013;88:256–66.
- [8] Su A, Liang Y, Zhao O. Experimental and numerical studies of S960 ultra-high strength steel welded I-section columns. *Thin Walled Struct* 2021;159:107166.
- [9] Su A, Zhao O. Experimental and numerical investigations of S960 ultra-high strength steel slender welded I-section columns failing by local–flexural interactive buckling. *Thin Walled Struct* 2022;180:109898.
- [10] Wang F, Liang Y, Zhao O. Experimental and numerical studies of pin-ended press-braked S960 ultra-high strength steel channel section columns. *Eng Struct* 2020;215:110629.
- [11] Su A, Yang H, Wang Y, Zhao O. Experimental and numerical investigations of S960 ultra-high strength steel welded I-section beams with in-plane flexural failure. *Thin Walled Struct* 2023;190:110969.
- [12] Wang F, Zhao O, Young B. Flexural behaviour and strengths of press-braked S960 ultra-high strength steel channel section beams. *Eng Struct* 2019;200:109735.
- [13] Bradford M, Liu X. Flexural-torsional buckling of high strength steel beams. *J Constr Steel Res* 2016;124:122–31.
- [14] Wang Y, Bradford M, Liu X. Strength design of welded high-strength steel beams considering coupled local and global buckling. *Thin Walled Struct* 2020;149:106391.
- [15] Su A, Sun Y, Liang Y, Zhao O. Experimental and numerical studies of S960 ultra-high strength steel welded I-sections under combined compression and minor-axis bending. *Eng Struct* 2021;243:112675.
- [16] Su A, Sun Y, Zhao O, Liang Y. Local buckling of S960 ultra-high strength steel welded I-sections subjected to combined compression and major-axis bending. *Eng Struct* 2021;248:113213.

- [17] Su A, Wang Y, Rasmussen KJR, Gardner L. Structural performance and design of S960 ultra-high strength steel non-slender welded I-sections subjected to combined loading. *Eng Struct* 2023;293:116593.
- [18] Su A, Jiang K, Wang M, Zhao O. S960 ultra-high strength steel slender welded I-section beam-columns: Testing, numerical modelling and design. *Thin Walled Struct* 2022;177:109452.
- [19] Fang H, Chan T. Buckling resistance of welded high-strength-steel box-section members under combined compression and bending. *J Constr Steel Res* 2019;162:105711.
- [20] EN 1993-1-12:2007, Eurocode 3: design of steel structures – Part 1–12: Additional rules for the extension of EN 1993 up to grades S700, European Committee for Standardization (CEN), Brussels, 2007.
- [21] ANSI/AISC 360–16, Specification for the Structural Steel Buildings, American Institute of Steel Construction (AISC), 2016.
- [22] K. Hibbitt, I. Sorensen, ABAQUS/Standard user's Manual Volumes I-III and ABAQUS CAE Manual. Version 6.14. Pawtucket (USA), 2014.
- [23] Kyvelou P, Gardner L, Nethercot DA. Finite element modelling of composite cold-formed steel flooring systems. *Eng Struct* 2018;158:28–42.
- [24] Yan J, Feng J, Luo Y, Du Y. Compression behaviour of stainless-steel square tubular columns at cold-region low temperatures. *J Constr Steel Res* 2021;187:106984.
- [25] Zhou X, Chen Y, Ke K, Yam MCH, Li H. Hybrid steel staggered truss frame (SSTF): a probabilistic spectral energy modification coefficient surface model for damage-control evaluation and performance insights. *J Build Eng* 2022;45:103556.
- [26] Su A, Jiang K, Wang Y, Zhao O. Experimental and numerical investigations of S960 ultra-high strength steel welded I-section stub columns after exposure to elevated temperatures. *Thin Walled Struct* 2023;183:110349.
- [27] Chen M, Young B. Structural behavior of cold-formed steel semi-oval hollow section beams. *Eng Struct* 2019;185:400–11.
- [28] Su A, Sun Y, Liang Y, Zhao O. Material properties and membrane residual stresses of S690 high strength steel welded I-sections after exposure to elevated temperatures. *Thin Walled Struct* 2020;152:106723.
- [29] Yun X, Zhu Y, Meng X, Gardner L. Welded steel I-section columns: residual stresses, testing, simulation and design. *Eng Struct* 2023;282:115631.
- [30] Yuan H, Wang Y, Gardner L, Shi Y. Local-overall interactive buckling of welded stainless box section compression members. *Eng Struct* 2014;67:62–76.
- [31] Wang J, Gardner L. Flexural buckling of hot-finished high strength steel SHS and RHS columns. *J Struct Eng* 2017;143:04017028.
- [32] Su A, Jiang K, Liang Y, Zhao O. Post-fire behaviour and resistances of S690 high strength steel welded I-section stub columns. *Thin Walled Struct* 2021;169:108422.
- [33] Wang Y, Yang H, Wang Y, Su A. Testing, numerical modelling and design of S890 and S960 ultra-high strength steel circular hollow sections under combined loading. *Thin Walled Struct* 2023;192:111102.
- [34] Meng X, Gardner L. Flexural buckling of normal and high strength steel CHS columns. *Structures* 2021;34:4364–75.
- [35] Sun Y, Liang Y, Su A, Zhao O. Local-flexural interactive buckling of S690 high strength steel slender welded I-section columns: testing, modelling and design. *Thin Walled Struct* 2021;166:108064.
- [36] Zhao O, Gardner L, Young B. Behaviour and design of stainless steel SHS and RHS beam-columns. *Thin Walled Struct* 2016;106:330–45.
- [37] Zhao O, Rossi B, Gardner L, Young B. Experimental and numerical studies of ferritic stainless steel cross-sections under combined compression and bending. *J Struct Eng* 2016;142(2):04015110.
- [38] EN 1993-1-1: 2005, Eurocode 3: Design of steel structures – part 1–1: General Rules and Rules for Buildings, European Committee for Standardization (CEN), Brussels, 2005.
- [39] Meng X, Gardner L, Sadowski A, Rotter J. Elasto-plastic behaviour and design of semi-compact circular hollow sections. *Thin Walled Struct* 2020;148:106486.
- [40] A. Taras, R. Greiner, H. Unterweger, Proposal for amended rules for member buckling and semi-compact cross-section design, Institute of Steel Structures, Graz University of Technology, 2013.
- [41] Gardner L, Yun X, Walport F. The continuous strength method – review and outlook. *Eng Struct* 2023;275:114924.
- [42] Liew A, Gardner L. Ultimate capacity of structural steel cross-sections under compression, bending and combined loading. *Structures* 2015;1:2–11.
- [43] Gardner L, Yun X, Fieber A, Macorini L. Steel design by advanced analysis: material modeling and strain limits. *Engineering* 2019;5(2):243249.
- [44] Yun X, Gardner L, Boissonnade N. The continuous strength method for the design of hot-rolled steel cross-sections. *Eng Struct* 2018;157:179–91.
- [45] Gardner L, Fieber A, Macorini L. Formulae for calculating elastic local buckling stresses of full structural cross-sections. *Structures* 2019;17:2–20.
- [46] Arrayago I, Real E, Mirambell E, Gardner L. The continuous strength method for the design of stainless steel hollow section columns. *Thin Walled Struct* 2020;154:106825.
- [47] Arrayago I, Real E, Gardner L, Mirambell E. The continuous strength method for the design of stainless steel hollow section beam-columns. *Eng Struct* 2021;238:111981.
- [48] EN 1990:2002+A1:2005, Eurocode – Basis of structural design. European Committee for Standardization (CEN), Brussels, 2005.
- [49] Wang J, Afshan S, Gkantou M, Theofanous M, Baniotopoulos C, Gardner L. Flexural behaviour of hot-finished high strength steel square and rectangular hollow sections. *J Constr Steel Res* 2016;121:97–109.
- [50] Byfield MP, Nethercot DA. Material and geometric properties of structural steel for use in design. *Struct Eng* 1997;75(21):363–7.