Experimental and numerical studies of ferritic stainless steel tubular cross-sections under combined compression and bending

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Abstract: An experimental and numerical study of ferritic stainless steel tubular cross-sections under combined loading is presented in this paper. Two square hollow section (SHS) sizes – SHS 40×40×2 and SHS 50×50×2 made of grade EN 1.4509 (AISI 441) stainless steel were considered in the experimental program, which included two concentrically-loaded stub column tests, two four-point bending tests and fourteen eccentrically-loaded stub column tests. In parallel with the experimental investigation, a finite element (FE) study was also conducted. Following validation of the FE models against the test results, parametric analyses were carried out to generate further structural performance data. The experimental and numerical results were analyzed and compared with the design strengths predicted by the current European stainless steel design code EN 1993-1-4 and American stainless steel design specification SEI/ASCE-8. The comparisons revealed that the codified capacity predictions for ferritic stainless steel cross-sections under combined loading are unduly conservative. The deformation-based Continuous Strength Method (CSM) has been extended to cover the case of combined loading. The applicability of CSM to the design of ferritic stainless steel cross-sections under combined loading was also evaluated; The CSM was shown to offer

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substantial improvements in design efficiency over existing codified methods. Finally, the reliability of the proposals was confirmed by means of statistical analyses according to both the SEI/ASCE-8 requirements and those of EN 1990.

Introduction

Ferritic stainless steels are becoming an increasingly attractive choice in a range of engineering applications due to their unique combination of moderate material price, favorable mechanical properties and resistance to corrosion (Cashell and Baddoo 2014). The initial material cost of stainless steel is largely dominated by the nickel content. Compared to their austenitic and duplex counterparts, the ferritic grades have no or very low nickel content and thus relatively low material price. Previous relevant studies on ferritic stainless steels are briefly reviewed herein. Hyttinen (1994) performed a series of eccentric compression tests on tubular members to investigate the interactive buckling behavior of ferritic stainless steel beam-columns and to assess the accuracy of the European code and American specification. Van den Berg (2000) collected previous test data on ferritic stainless steel open sections and studied the flexural-torsional buckling behavior of I-section columns and the lateral-torsional buckling behavior of lipped channel beams. A series of 48 concentric compression tests on ferritic stainless steel lipped channel sections were carried out by Rossi et al. (2010) to investigate their combined distortional and overall flexural-torsional buckling behavior. Afshan and Gardner (2013a) conducted a comprehensive experimental program on square hollow section (SHS) and rectangular hollow section (RHS) structural members, including stub column tests, beam tests and flexural buckling tests, to verify the basic structural performance of ferritic stainless steel elements. The web crippling behavior of ferritic stainless steel SHS and RHS, strengthened by fiber reinforced polymers, were tested and

analyzed by Islam and Young (2012, 2013). However, to date, there have been no investigations into the cross-sectional behavior of ferritic stainless steel sections under combined loading, and this is therefore the focus of the present study.

Firstly, an experimental study was carried out, including tests on two concentrically-loaded stub columns, two beams, ten stub columns loaded at an eccentricity to one principal axis and four stub columns loaded at eccentricities to both principal axes. The experimental results were supplemented by numerically FE generated data. The FE models were initially validated against the test results and then utilized to perform parametric studies to generate further data over a wider range of cross-section slenderness and combinations of loading. The experimental and numerical data were used to assess the accuracy of the codified design provisions given in EN 1993-1-4 (CEN 2006) and SEI/ASCE-8 (ASCE 2002). Furthermore, the applicability and reliability of new design proposals (Liew and Gardner 2014; Zhao et al. 2015a), which were derived through extension of the CSM to the case of stainless steel cross-sections under combined loading, were carefully evaluated.

Experimental investigation

General

An experimental program was conducted at the University of Liège and Imperial College London, to investigate the behavior of cold-formed grade EN 1.4509 (AISI 441) ferritic stainless steel tubular sections under combined axial load and bending moment. The two studied cross-sections were SHS 40×40×2 and SHS 50×50×2, which are class 1 and class 3, respectively, according to the slenderness limits stated in EN 1993-1-4 (CEN 2006). The chemical compositions for each section, as provided by the mill certificates, are shown in

Table 1. The test specimens were labeled using a number and a letter; the number identifies the section sizes with ‘1’ for SHS 40×40×2 and ‘2’ for SHS 50×50×2, while the letter designates the test type as follows: A indicates a stub column under pure compression, B identifies a beam, C-G signify stub columns under uniaxial bending plus compression and H-I are stub columns under biaxial bending plus compression.

Material testing and geometric imperfection measurements

Prior to structural testing, tensile coupon tests and geometric imperfection measurements were conducted. The detailed procedures and experimental setup for the coupon tests are described by Afshan et al. (2013a), while only a brief summary of the key test results is reported herein. For each cross-section, the average measured flat and corner material properties are summarized in Tables 2 and 3, respectively, where E is the Young’s modulus, $\sigma_{0.2}$ is the 0.2% proof stress, $\sigma_{1.0}$ is the 1.0% proof stress, σ_u is the ultimate tensile strength, ε_u is the strain at the ultimate tensile stress, ε_f is the plastic strain after fracture measured over the standard gauge length (40 mm for flat coupons and 25 mm for corner coupons), and n , $n'_{0.2,1.0}$ and $n'_{0.2,u}$ are the strain hardening exponents used in the compound Ramberg–Osgood (R–O) material model (Ramberg and Osgood 1943; Hill 1944; Mirambell and Real 2000; Rasmussen 2003; Gardner and Ashraf 2006). Owing to the relatively short specimen length, global buckling was insignificant, hence only local geometric imperfections were measured, following the procedures and test setup employed by Schafer and Peköz (1998). For each specimen, imperfection measurements were conducted along the centerlines of all four faces of the cross-section, but only over the central 50% of the member length to eliminate the effect of end flaring (Cruise and Gardner 2006) and welds (Zhao et al. 2015b). The maximum imperfection amplitude for each face was defined as the maximum deviation

from a linear regression line fitted to the data set. The largest value of the maximum measured deviation from all four faces was taken as the initial local geometric imperfection amplitude of the specimen ω_0 . Note that all the following calculations are based on the measured material and geometric properties.

Stub column tests

Two concentrically-loaded stub column tests were performed to obtain the cross-sectional load-carrying and deformation capacities under pure compression. The nominal length for each specimen complies with the guidelines of Ziemian (2010). The measured geometric dimensions and imperfection amplitudes of the stub columns are reported in Table 4, where L is the member length, B and H are the outer cross-section width and depth, respectively, t is the material thickness, r_i is the internal corner radius, A is the cross-section area and ω_0 is the measured maximum local geometric imperfection. The stub columns were compressed using a Schenck 600 kN hydraulic testing machine with fixed end platens, at a constant rate of 0.1 mm/min. Fig. 1 depicts the stub column test setup, consisting of four LVDTs to determine the end shortening and four strain gauges, attached at mid-height of each specimen, to measure the longitudinal strains. The applied load, end shortening measurements and strain gauge readings were recorded using the data acquisition equipment ScanWin at one-second intervals. The true end-shortening values were obtained by eliminating the elastic deformation of the end platens of the testing machine from the end-shortening measurements on the basis of the strain gauge readings (Centre for Advanced Structural Engineering 1990). The modified true load–end shortening curves are presented in Fig. 2, while the key test results, including the ultimate load N_u , the end shortening δ_u at ultimate load and the ratio of $N_u / A\sigma_{0.2}$ are reported in Table 5. The deformed specimens SHS 40×40×2-1A and SHS

50×50×2-2A are shown in Figs 3(a) and 3(b), exhibiting elephant foot and local buckling failure modes, respectively.

Four-point bending tests

For each cross-section, a symmetric four-point bending test was carried out to investigate the flexural performance and rotation capacity of ferritic stainless steel sections under constant bending moment. The geometric properties and initial geometric imperfections were carefully measured prior to testing, and are listed in Table 6. Both of the specimens had a total length of 700 mm and a length between the loading points of 200 mm. Two steel rollers, placed 50 mm inward from the ends of the beams, were used to provide simple supports to the specimens with longitudinal displacement fixed at one end, resulting in a flexural span length of 600 mm. The beams were loaded symmetrically at two points allowing free rotation and longitudinal displacement through a spreader beam (see Fig. 4) at a constant loading rate of 0.5 mm/min. Strain gauge readings confirmed that no net axial force was generated in the beams. Wooden blocks were inserted into the tube at the loading points to prevent web crippling. Three string potentiometers were located at mid-span and at the two loading points to measure the respective vertical deflections. Four strain gauges were affixed to the top and bottom flanges of the specimens at mid-span to determine the extreme compressive and tensile strains. The data acquisition system DATASCAN was utilized to record the applied load, vertical deflections and strains at one-second intervals.

Both of the beam specimens failed by local buckling of the compression flange and upper portion of the webs within the constant moment region; Fig. 5 depicts the failure mode of the beam specimen SHS 40×40×2-1B. Table 7 reports the key test results, including the test

ultimate moment M_u , the ratios of test ultimate moment to the elastic and plastic moment capacities (M_u / M_{el} and M_u / M_{pl}), where M_{el} and M_{pl} are the cross-section elastic and plastic moment capacities, equal to the measured 0.2% proof stress multiplied by the elastic and plastic section moduli (determined from the measured geometry), respectively, and the rotation capacity R , which was determined from Eq. (1), where κ_{pl} is the elastic component of the total curvature corresponding to M_{pl} and κ_u is the curvature at which the moment–curvature curve falls back below M_{pl} . The normalized experimental moment–curvature curves $M / M_{pl} - \kappa / \kappa_{pl}$ are shown in Fig. 6, where κ is the curvature approximated according to Eq. (2) (Chan and Gardner 2008), in which D_M is the vertical deflection at mid-span, D_L is the average vertical deflection at the two loading points and L_m is the length between the two loading points (200 mm).

$$R = \frac{\kappa_u}{\kappa_{pl}} - 1 \quad (1)$$

$$\kappa = \frac{8(D_M - D_L)}{4(D_M - D_L)^2 + L_m^2} \quad (2)$$

Eccentrically-loaded stub column tests

For each cross-section, seven eccentrically-loaded stub column tests were conducted, with the aim of investigating the cross-sectional behavior of ferritic stainless steel tubular sections under combined axial compression and bending. The nominal length of each specimen was equal to that of the corresponding stub column specimen. Measurements of the geometric properties were performed, before 15 mm thick end plates were welded to the specimen ends.

The measured geometric properties and imperfection amplitudes are listed in Tables 8 and 10 for specimens tested under uniaxial and biaxial eccentric compression, respectively.

The combined loading tests were performed using a Zwick/Roell 600 kN hydraulic testing machine with hemispherical bearings at both ends to provide pin-ended boundary conditions. The specimens were eccentrically bolted to the hemispherical bearings and the initial eccentricities were varied to provide a range of bending moment-to-axial load ratios. Since the center of rotation of the hemispherical bearing is located at the centroid of its flat face, the effective length L_e in each test was equal to the specimen length plus the thickness of the two welded end plates. Fig. 7(a) and Fig. 7(b) depict a photograph and a schematic diagram of the test setup, respectively. The instrumentation consisted of two inclinometers (one at each end of the specimens) to measure the end rotations, four strain gauges attached to the extreme fibers of the cross-sections at mid-height to obtain the maximum and minimum longitudinal strains, and two LVDTs located along both principal axes to determine the generated lateral deflections and thus the second order bending moments (Fujimoto et al. 2004; Gardner et al. 2011; Zhao et al. 2015b). Finally, the applied load, lateral deflections, end rotations and longitudinal strains were recorded using the data acquisition system ScanWin at a rate of one-second.

Table 9 reports the key experimental results for the uniaxial eccentrically-loaded stub column tests, including the failure load N_u , the initial loading eccentricity e_0 , the generated lateral deflection at the failure load e' , the failure moment $M_u = N_u(e_0 + e')$ and the corresponding end rotation at the failure load ϕ_u , while the experimental results from the biaxial eccentrically-loaded stub column tests are fully reported in Table 11. Note that the initial loading eccentricities reported in Tables 8–11 are not the nominal but the actual values,

which are calculated on the basis of the strain gauge readings, following the derivation procedures of Zhao et al. (2015b). The experimental load–end rotation curves are shown in Figs 8 and 9 respectively for specimens subjected to uniaxial eccentric compression and biaxial eccentric compression. Typical local buckling failure modes from the combined loading tests are depicted in Figs 10 and 11, in which the welded end plates have been cut from the specimens after the combined loading tests.

Numerical modeling

General

A numerical modeling study, adopting the general-purpose finite element analysis package ABAQUS (2012), was performed in parallel with the laboratory testing program. The FE simulations were carried out initially to replicate the full experimental load–deformation histories and then used to perform parametric studies to generate more data over a wider range of cross-section slenderness and combinations of loading.

Basic modeling assumptions

Having been successfully employed in previous studies (Gardner and Nethercot 2004b; Theofanous and Gardner 2009; Theofanous et al. 2009; Huang and Young 2013; Zhao et al. 2015a) concerning the modeling of thin-walled structures, the four-noded doubly curved shell element with reduced integration, S4R (ABAQUS 2012), was selected as the element type throughout the present numerical investigation. An element size equal to the cross-section thickness was utilized in the flat regions of the modeled cross-sections, while a finer mesh of 4 elements was assigned to the corners to accurately discretize the curved corner geometry.

The measured stress-strain curves were firstly represented by the two-stage Ramberg–Osgood material model (Mirambell and Real 2000; Gardner and Ashraf 2006) and then converted into the format of true stress and log plastic strain according to Eq. (3) and Eq. (4), as required by ABAQUS.

$$\sigma_{true} = \sigma_{nom} (1 + \varepsilon_{nom}) \quad (3)$$

$$\varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (4)$$

in which σ_{true} is the true stress, ε_{ln}^{pl} is the log plastic strain, and σ_{nom} and ε_{nom} are the engineering stress and strain, respectively.

Following the recommendations of Ashraf et al. (2006) and Cruise and Gardner (2008a), the corner material properties were assigned not only to the curved corner regions of the FE models but also to the adjacent flat regions extending to a distance of two times the cross-section thickness beyond the corners, since strength enhancement due to the cold-rolling process is also observed here. Two types of residual stresses (bending and membrane residual stresses) exist within cold-formed stainless steel members. However, the magnitude of the membrane residual stresses is small compared to that of the through-thickness bending residual stresses (Cruise and Gardner 2008b; Jandera et al. 2008; Huang and Young 2012). In addition, the effect of the through-thickness bending residual stress is inherently included in the measured material properties due to straightening of the initially longitudinally curved coupons during tensile testing (Rasmussen and Hancock 1993a). Residual stresses were therefore not explicitly incorporated into the developed FE models.

Symmetry was exploited by modeling only half the member length and then applying suitable symmetry boundary conditions at the mid-height of the models. The following end section

boundary conditions were employed: For the concentrically loaded stub column FE models, all degrees of freedom at the loaded end section were coupled with a concentric reference point, only allowing longitudinal translation, in order to model fixed ended boundary conditions. For the eccentrically loaded stub column FE models, the end section was coupled to an eccentric reference point, where all degrees of freedom were restrained except for the longitudinal translation and rotation about the axis of buckling, to simulate pin-ended boundary conditions. In addition, the eccentric reference point was offset longitudinally from the loaded end section by a distance equal to the thickness of the welded end plate (15mm) to accurately model the effective member length. Similar end section boundary conditions to those used for the eccentrically-loaded stub columns were applied to the four-point bending FE models, with the only difference being that the reference point was located at the mid-point of the bottom flange, which simulated the simply-supported boundary conditions employed in the four-point bending beam tests.

Initial local geometric imperfections were incorporated into the FE models in the form of the lowest elastic local buckling mode shape, determined by means of a prior eigenvalue buckling analysis. Three imperfection amplitudes were utilized to factor the buckling mode pattern, including the measured maximum imperfection amplitude w_0 , 1/100 of the cross-section thickness and the imperfection amplitude predicted by the modified Dawson and Walker (D&W) model $w_{D\&W}$ (Dawson and Walker 1972; Gardner and Nethercot 2004b), as given by Eq. (5), in which $\sigma_{cr,min}$ is the minimum elastic buckling stress of all the plate elements making up the cross-section. Upon incorporation of the initial geometric imperfections, a nonlinear static Riks analysis was performed to trace the full load–deformation histories of the FE models.

$$\omega_{D\&W} = 0.023 \left(\frac{\sigma_{0.2}}{\sigma_{cr,\min}} \right) t \quad (5)$$

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269 *Validation of numerical models*

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271 The FE failure loads for the various imperfection amplitudes are compared with the
 272 corresponding experimental results in Table 12, revealing that the models with the measured
 273 imperfection values yield the highest accuracy and consistency in the prediction of failure
 274 loads, with the mean ratio of FE to test failure loads equal to 0.97 and the corresponding
 275 coefficient of variation (COV) of 0.03. The slight conservatism may be due to the use of the
 276 tensile coupon material properties for both the compression and tension portions of the FE
 277 models, since, as indicated by Rasmussen and Hancock (1993a), the stress–strain curve of
 278 stainless steel in tension is marginally lower than that in compression. More accurate FE
 279 failure load predictions may be obtained if the compressive and tensile material properties
 280 were assigned to the relevant portions of the FE models, though reliable measurement of the
 281 stress–strain characteristics of thin-walled sections is not straightforward. The modified
 282 Dawson and Walker model may also be seen to result in very accurate predictions.
 283 Comparisons between the experimental and numerical load–end shortening, normalized
 284 moment–curvature and load–end rotation curves for typical tested specimens are displayed in
 285 Figs 12–15, where the solid and dashed lines represent the test and FE curves, respectively.
 286 These typical comparisons show that the full experimental loading histories have been
 287 accurately replicated by the FE simulations. Excellent agreement is also obtained between the
 288 test and numerical local buckling failure modes, as depicted in Figs 16–19. In summary, the
 289 FE models have been shown to be capable of replicating the key test results, full experimental
 290 load–deformation histories and observed failure modes.

Parametric studies

Upon validation of the FE models, a series of parametric studies was carried out to generate further structural performance data over a wider range of cross-section slenderness and combinations of loading. In the parametric studies, the average material properties from the tensile coupon tests were adopted and the incorporated initial local imperfection amplitudes were predicted by the modified Dawson and Walker model. All the modeled cross-sections had an outer width of 100 mm and an outer depth of either 100 mm or 150 mm. The length of each stub column or combined loading FE model was set to be equal to four times its mean outer dimensions, while the length of each beam model was set equal to 15 times its mean outer dimensions. The cross-section thickness was varied from 3.4 mm to 10 mm and the initial loading eccentricities ranged between 3 mm and 250 mm, resulting in a wide range of cross-section slenderness and loading combinations being considered. The modeled cross-sections cover all the four classes according to the slenderness limits in EN 1993-1-4 (CEN 2006). The parametric study results are analyzed and discussed in the following section.

Discussion and assessment of current design methods

General

In this section, four methods for the design of stainless steel cross-sections under combined loading, including two codified methods – EN 1993-1-4 (CEN 2006) and SEI/ASCE-8 (ASCE 2002) and two recently proposed deformation-based design approaches – the Continuous Strength Method (CSM) (Liew and Gardner 2014) and simplified CSM (Zhao et

al. 2015a), are described. The accuracy of each method is then assessed against the generated test and FE results. Tables 13 and 14 provide ratios of the test (or FE) to predicted capacities, calculated in terms of the axial load, $N_u / N_{u,pred}$, in which N_u is the test (or FE) axial load corresponding to the distance on the N–M interaction curve (or surface) from the origin to the test (or FE) data point, while $N_{u,pred}$ is the predicted axial load corresponding to the distance from the origin to the intersection with the design interaction curve (or surface), assuming proportional loading. An example showing the determination of N_u and $N_{u,pred}$ from a uniaxial bending plus compression interaction curve is illustrated in Fig. 20. A value of $N_u / N_{u,pred}$ greater than unity indicates that the test (or FE) data point lies outside the interaction curve and is safely predicted. Note that all comparisons have been made based on the measured material and geometric properties and that all partial factors have been set equal to unity.

European code EN 1993-1-4 (EC3)

For cross-sectional capacity under combined loading, the current European code for stainless steel, EN 1993-1-4 (CEN 2006) adopts the same provisions as those given in EN 1993-1-1 (CEN 2005) for carbon steel, in which the interaction formula for class 3 cross-sections is derived based on a linear elastic response, as given by Eq. (6), where N_{Ed} is the design ultimate load, and $M_{Ed,y} = N_{Ed} (e_{0y} + e'_y)$ and $M_{Ed,z} = N_{Ed} (e_{0z} + e'_z)$ are the design bending moments about the two principal axes. For class 4 cross-sections, the effective cross-section properties replace the gross cross-section properties in Eq. (6).

$$\frac{N_{Ed}}{A\sigma_{0.2}} + \frac{M_{Ed,y}}{M_{el,y}} + \frac{M_{Ed,z}}{M_{el,z}} \leq 1 \quad (6)$$

The interaction formulae for class 1 and class 2 cross-sections were derived on the assumption of full plasticity throughout the cross-section at failure, as given by Eqs (7) and (8) for RHS under major axis bending plus compression and minor axis bending plus compression, respectively, and Eq. (9) for RHS subjected to biaxial bending plus compression, in which $M_{R,y}$ and $M_{R,z}$ are respectively the reduced plastic moment capacities about the major and minor axes due to the existence of the axial force N_{Ed} , n is equal to $N_{Ed} / A\sigma_{0.2}$, a_w and a_f are the ratios of the web area A_w and flange area A_f to gross cross-section area A , respectively, and α_e and β_e , which are equal to $1.66 / (1 - 1.13n^2)$, are the interaction coefficients for biaxial bending.

$$M_{Ed,y} \leq M_{R,y} = M_{pl,y} \frac{(1-n)}{(1-0.5a_w)} \leq M_{pl,y} \quad (7)$$

$$M_{Ed,z} \leq M_{R,z} = M_{pl,z} \frac{(1-n)}{(1-0.5a_f)} \leq M_{pl,z} \quad (8)$$

$$\left[\frac{M_{Ed,y}}{M_{R,y}} \right]^{\alpha_e} + \left[\frac{M_{Ed,z}}{M_{R,z}} \right]^{\beta_e} \leq 1 \quad (9)$$

The accuracy of EN 1993-1-4 (CEN 2006) is assessed by comparing the experimental and numerical results with the EC3 predicted capacities. As reported in Tables 13 and 14, the mean values of $N_u / N_{u,EC3}$ ratio are equal to 1.18 with a COV equal to 11.0% and 1.24 with a COV of 18.3% for RHS subjected to uniaxial and biaxial bending plus compression, respectively, indicating unduly scattered strength predictions although with reasonable accuracy, on average.

American Specification SEI/ASCE-8

The American Specification SEI/ASCE-8 (ASCE 2002) uses the same interaction formula for both the cross-sectional and global behavior of stainless steel elements under combined loading, as given by Eq. (10). However, as discussed by Zhao et al. (2015a), for a short beam-column under constant first order bending moment, the equivalent moment factors (C_{my} and C_{mz}) and the magnification factors (α_{ny} and α_{nz}) are all approximately equal to unity. Thus, Eq. (10) reduces to the linear interaction formula given by Eq. (11), in which M_{ny} and M_{nz} are the codified bending resistances calculated based on the inelastic reserve capacity according to clause 3.3.1.1 of SEI/ASCE-8 (ASCE 2002).

$$\frac{N_{Ed}}{A\sigma_{0.2}} + \frac{C_{my}M_{Ed,y}}{M_{ny}\alpha_{ny}} + \frac{C_{mz}M_{Ed,z}}{M_{nz}\alpha_{nz}} = 1 \quad (10)$$

$$\frac{N_{Ed}}{A\sigma_{0.2}} + \frac{M_{Ed,y}}{M_{ny}} + \frac{M_{Ed,z}}{M_{nz}} = 1 \quad (11)$$

A comparison of the test and FE results with the American Specification and European code is shown in Fig. 21, where the ratio of test (or FE) capacity to predicted capacity $N_u / N_{u,pred}$ is plotted against the cross-section slenderness $\bar{\lambda}_p = \sqrt{\sigma_{0.2} / \sigma_{cr}}$, in which σ_{cr} is the elastic buckling stress of the cross-section allowing for element interaction (Schafer and Ádány 2006; Theofanous and Gardner 2012) under the applied loading conditions; for stub columns and beams, the elastic buckling stresses are calculated under pure compression and bending respectively, while for stub columns under combined loadings, σ_{cr} is determined under the actual combination of compression and bending. Compared to the EC3 predictions, the ASCE design strengths are more conservative in the low cross-section slenderness range (i.e. class 1 and class 2 cross-sections) but more accurate for higher $\bar{\lambda}_p$ values (i.e. class 3 and class 4

cross-sections). The mean $N_u / N_{u,ASCE}$ ratios, as given in Tables 13–14, are 1.23 and 1.49 for uniaxial bending plus compression and biaxial bending plus compression, respectively, and the corresponding values of COV are 6.4% and 8.7%, revealing overall more conservative but less scattered strength predictions by SEI/ASCE-8 (ASCE 2002) than by EN 1993-1-4 (CEN 2006). Research by Rasmussen and Hancock (1993b) has shown that stainless steel tubular cross-sections can reach their full plastic moment capacities provided that the flat width-to-thickness ratio is less than a specified slenderness limit. Use of the full plastic moment capacities in Eq (11) for these cross-sections was assessed. The results showed that, on this basis, the mean ratios of the test (or FE) to predicted capacities reduced to 1.21 and 1.48 for uniaxial bending plus compression and biaxial bending plus compression cases, respectively, and the COVs decreased to 6.0% and 8.4%, confirming the suitability of the proposal.

Continuous Strength Method (CSM)

The Continuous Strength Method (CSM) is a deformation-based design approach (Gardner 2008; Gardner et al. 2011; Afshan and Gardner 2013b; AISC 2013), which relates the strength of a cross-section to its deformation capacity and employs a bi-linear material model to consider strain hardening. The bi-linear material model in the CSM was previously developed based on the material data from austenitic and duplex stainless steels, and has recently been extended to cover ferritic stainless steel grades by Bock et al. (2015). The relationship between the deformation capacity, expressed in terms of the strain ratio $\varepsilon_{csm} / \varepsilon_y$, and the cross-section slenderness $\bar{\lambda}_p$ is defined by Eq. (12), where ε_{csm} is the maximum attainable strain of the cross-section, $\varepsilon_y = \sigma_{0.2} / E$ is the yield strain and

407 $\varepsilon_u = 0.6 - 0.6\sigma_{0.2} / \sigma_u$ is used to approximate the strain at the material ultimate strength. Eq.
 408 (12) applies for cross-section slenderness values less than or equal to 0.68, which is the
 409 current limit of applicability of the CSM, though further work is underway to extend the
 410 CSM to also cover slender cross-sections.

$$411 \quad \frac{\varepsilon_{csm}}{\varepsilon_y} = \frac{0.25}{\bar{\lambda}_p^{3.6}} \text{ but } \leq \min\left(15, \frac{0.4\varepsilon_u}{\varepsilon_y}\right) \quad (12)$$

412 The strain hardening modulus E_{sh} , employed in the CSM elastic, linear hardening material
 413 model (see Fig. 22), may be calculated for ferritic stainless steel from Eq. (13).

$$414 \quad E_{sh} = \frac{\sigma_u - \sigma_{0.2}}{0.45\varepsilon_u - \varepsilon_y} \text{ if } \frac{\varepsilon_y}{\varepsilon_u} < 0.45, \text{ else } E_{sh} = 0 \quad (13)$$

415 The CSM design stress σ_{csm} can be found from Eq. (14), while the CSM resistances for RHS
 416 subjected to pure compression and pure bending are determined from Eqs (15) and (16),
 417 respectively (Gardner et al. 2011; Afshan and Gardner 2013b; AISC 2013), where α is equal
 418 to 2 for RHS in bending about either principal axis.

$$419 \quad \sigma_{csm} = \sigma_{0.2} + E_{sh}(\varepsilon_{csm} - \varepsilon_y) \quad (14)$$

$$420 \quad N_{csm} = A\sigma_{csm} \quad (15)$$

$$421 \quad M_{csm} = M_{pl} \left[1 + \frac{E_{sh}}{E} \frac{W_{el}}{W_{pl}} \left(\frac{\varepsilon_{csm}}{\varepsilon_y} - 1 \right) - \left(1 - \frac{W_{el}}{W_{pl}} \right) / \left(\frac{\varepsilon_{csm}}{\varepsilon_y} \right)^\alpha \right] \quad (16)$$

422 Based on the assumption of a linear through-depth strain distribution and the bi-linear
 423 material model, the CSM resistances for RHS under various combined loading cases,
 424 including major axis bending plus compression, minor axis bending plus compression and
 425 biaxial bending plus compression, can be derived by integration and represented through
 426 interaction expressions, as shown in Eqs (17)–(19) (Liew and Gardner 2014), respectively:

$$M_{Ed,y} \leq M_{R,csml,y} = M_{csm,y} \left[1 - \left(\frac{N_{Ed}}{N_{csm}} \right)^{\alpha_y} \right]^{1/b_y} \quad (17)$$

$$M_{Ed,z} \leq M_{R,csml,z} = M_{csm,z} \left[1 - \left(\frac{N_{Ed}}{N_{csm}} \right)^{\alpha_z} \right]^{1/b_z} \quad (18)$$

$$\left(\frac{M_{Ed,y}}{M_{R,csml,y}} \right)^{\alpha_{csm1}} + \left(\frac{M_{Ed,z}}{M_{R,csml,z}} \right)^{\beta_{csm1}} \leq 1 \quad (19)$$

in which N_{csm} , $M_{csm,y}$ and $M_{csm,z}$ are the CSM compression and bending (major and minor axes) resistances, which act as the end points of the interaction curves and are calculated according to Eqs (15) and (16), and $M_{R,csml,y}$ and $M_{R,csml,z}$ are the reduced CSM bending resistances about the major and minor axes due to N_{Ed} ; note that the ‘1’ in the subscript signifies resistances determined on the basis of the proposals of Liew and Gardner (2014), where for strain ratios $\varepsilon_{csm} / \varepsilon_y$ greater than or equal to 3 (corresponding to cross-section slenderness $\bar{\lambda}_p$ less than about 0.5), the extent of reduction is determined by $\alpha_y = 1.2 + A_w / A$, $\alpha_z = 1.2 + A_f / A$ and $b_y = b_z = 0.8$, and the interaction coefficients for biaxial bending are defined by α_{csm1} and β_{csm1} , whose values are equal to $1.75 + W_r (2n_{csm}^2 - 0.15)$ and $1.6 + (3.5 - 1.5W_r)n_{csm}^2$, respectively, where W_r is the ratio of major to minor axis plastic section moduli $W_{pl,y} / W_{pl,z}$ and n_{csm} is the ratio of design axial force to CSM compression resistance N_{Ed} / N_{csm} , while for strain ratios $\varepsilon_{csm} / \varepsilon_y$ less than 3 (corresponding to $\bar{\lambda}_p$ greater than 0.5), all the above interaction parameters (α_y , α_z , b_y , b_z , α_{csm1} and β_{csm1}) are equal to unity. Fig. 23 shows typical CSM design interaction curves at the transition slenderness limit of 0.5, where the compression and bending end points are kept but with the non-linear interaction curve reducing to the linear interaction curve. Although a slight discontinuity and conservatism are induced, the linear interaction curve is proposed to

ensure compatibility with the increasingly elastic end points as the cross-section slenderness approaches $\bar{\lambda}_p = 0.68$, where the CSM axial and bending resistances are equal to $A\sigma_{0.2}$, $M_{el,y}$, and $M_{el,z}$.

The accuracy of the CSM was evaluated by comparing the test (or FE) capacity with the CSM predicted capacity; the results are reported in Tables 13–14. Overall, the CSM offers much more accurate and consistent predictions than the European code and American Specification, with the mean $N_u / N_{u,csml}$ ratios of 1.13 and 1.15 and the corresponding values of COV equal to 5.6% and 7.7% for stub columns under uniaxial eccentric compression and biaxial eccentric compression, respectively.

Simplified CSM

The simplified CSM was proposed by Zhao et al. (2015a) for the design of austenitic and lean duplex stainless steel cross-sections under combined loading. For cross-section slenderness $\bar{\lambda}_p$ less than or equal to 0.6, it was proposed to adopt the EC3 bi-linear interaction curves but anchored to the CSM end points for compression and bending resistances (N_{csm} , $M_{csm,y}$ and $M_{csm,z}$) rather than the yield load ($A\sigma_{0.2}$) and plastic ($M_{pl,y}$ and $M_{pl,z}$) bending resistances. The interaction formulae are given by Eqs (20)–(22) for RHS subjected to major axis, minor axis and biaxial eccentric compression, respectively, where $M_{R,csm,y}$ and $M_{R,csm,z}$ are the reduced CSM bending resistances about the major and minor axes, respectively, due to the presence of the axial load N_{Ed} , and $\alpha_{csm} = \beta_{csm} = 1.66 / (1 - 1.13n_{csm}^2)$ are the interaction coefficients for biaxial bending, which are taken from Eurocode 3 but based on the CSM end

points. For $\bar{\lambda}_p$ greater than 0.6, the linear interaction formula with CSM end points is proposed in order to ensure compatibility with the increasingly elastic end points, as given by Eq. (23). Typical nonlinear and linear design interaction curves at the transition slenderness limit are shown in Fig. 24.

$$M_{Ed,y} \leq M_{R,csm,y} = M_{csm,y} \frac{(1-n_{csm})}{(1-0.5a_w)} \leq M_{csm,y} \quad (20)$$

$$M_{Ed,z} \leq M_{R,csm,z} = M_{csm,z} \frac{(1-n_{csm})}{(1-0.5a_f)} \leq M_{csm,z} \quad (21)$$

$$\left[\frac{M_{Ed,y}}{M_{R,csm,y}} \right]^{\alpha_{csm}} + \left[\frac{M_{Ed,z}}{M_{R,csm,z}} \right]^{\beta_{csm}} \leq 1 \quad (22)$$

$$\frac{N_{Ed}}{N_{csm}} + \frac{M_{Ed,y}}{M_{csm,y}} + \frac{M_{Ed,z}}{M_{csm,z}} \leq 1 \quad (23)$$

A comparison of the test and FE results with the strength predictions of the simplified CSM is shown in Fig. 25, where the ratio of test (or FE) capacity to predicted capacity has been plotted against the cross-section slenderness $\bar{\lambda}_p$. The equivalent ratios based on the SEI/ASCE-8 predictions are also shown for comparison purposes. The results show that the simplified CSM yields substantially more accurate and less scattered strength predictions than SEI/ASCE-8 (ASCE 2002). A numerical evaluation of the simplified CSM is reported in Tables 13 and 14. The mean $N_u / N_{u,csm}$ ratios are equal to 1.07 and 1.06, with the corresponding COVs of 4.9% and 6.9%, for uniaxial bending plus compression and biaxial bending plus compression cases, respectively, revealing the highest accuracy in the prediction of ferritic stainless steel cross-section capacity under combined loading among the four design methods.

Reliability analysis

The reliability of the CSM and simplified CSM was assessed according to the requirements of both the Commentary to SEI/ASCE-8 (ASCE 2002) and Annex D of EN 1990 (CEN 2002). The experimental and numerical data considered in the reliability analyses are shown in Figs 26 and 27, where comparisons are made with the CSM and simplified CSM predicted resistances, respectively. SEI/ASCE-8 predictions are shown on both graphs for comparison purposes. The reliability analysis in SEI/ASCE-8 is derived based on a load combination of $1.2 \times \text{Dead Load} + 1.6 \times \text{Live Load}$ and a dead-to-live load ratio of 1:5. The statistical parameters, considering material and fabrication uncertainties, follow the recommendations made in SEI/ASCE-8 (ASCE 2002). Table 15 summarizes the key calculated statistical parameters, including the mean value (P_m) and COV (V_p) of the test-to-predicted capacity ratios, the resistance factor ϕ_0 , and the reliability index β_0 . A summary of the reliability analysis results for EN 1990 (CEN 2002) is reported in Table 16, where $k_{d,n}$ is the design (ultimate limit state) fractile factor, b is the average ratio of test (or FE) to design model resistance based on a least squares fit to all the data, V_δ is the COV of the tests and FE simulations relative to the resistance model, V_r is the combined COV incorporating both model and basic variable uncertainties, and γ_{M0} is the partial safety factor. The material overstrength and the variations in material strength and geometric properties follow the recommendations of Afshan et al. (2014). As can be seen from Tables 15 and 16, the required reliability indices are larger than the target value of 3.0 in SEI/ASCE-8 and the partial factors are less than the currently adopted value of 1.1 in EN 1993-1-4, both of which demonstrate the reliability of the new design proposals.

Summary

Overall, the American Specification SEI/ASCE-8 (ASCE 2002) results in the most conservative strength predictions among the four considered methods for the design of ferritic stainless steel cross-sections under combined loading, owing mainly to the use of linear interaction design curves. The European code EN 1993-1-4 (CEN 2006) generally leads to more accurate predictions than SEI/ASCE-8, but with increased scatter. As shown in Tables 13 and 14, the CSM (Liew and Gardner 2014) and simplified CSM (Zhao et al. 2015a) perform well for ferritic stainless steels and yield more accurate strength predictions with significantly lower scatter, compared to EN 1993-1-4 and SEI/ASCE-8.

Conclusions

A comprehensive experimental and numerical modeling program has been performed to investigate the structural behavior of ferritic stainless steel cross-sections under combined loading. A total of two concentrically-loaded stub column tests, two four-point bending tests and fourteen eccentrically-loaded stub column tests were conducted. The obtained experimental results were used to validate finite element models, which were subsequently employed to generate a series of parametric study results. The generated numerical data, together with the experimental results, were then utilized to assess the accuracy of four design methods, including two codified methods – EN 1993-1-4 (CEN 2006) and SEI/ASCE-8 (ASCE 2002), and two deformation-based design approaches – the CSM (Liew and Gardner 2014) and simplified CSM (Zhao et al. 2015a). Generally, the American Specification yielded the most conservative strength predictions among the four methods. The European code gave more accurate strength predictions than SEI/ASCE-8 on average, but the predictions were more scattered. The two deformation-based CSM design approaches

were shown to be well suited for application to ferritic stainless steel design, yielding a much higher level of accuracy and consistency in the prediction of cross-sectional resistances under combined loading, compared to the two codified methods. Finally, the reliability of the two CSM design proposals was confirmed by means of statistical analyses according to both SEI/ASCE-8 (ASCE 2002) and EN 1990 (CEN 2002). It is therefore recommended that the proposed approaches be considered for incorporation into future revisions of SEI/ASCE-8 and EN 1993-1-4 for stainless steel structures.

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Fig. 1. Stub column test setup.

Fig. 2. Load-end shortening curves for stub column tests.

(a) Elephant foot failure of specimen SHS 40x40x2-1A.

(b) Local buckling failure of specimen SHS 50x50x2-2A.

Fig. 3. Stub column failure modes.

Fig. 4. Four-point bending test setup.

Fig. 5. Four-point bending failure mode of specimen SHS 40×40×2-1B.

Fig. 6. Normalized moment–curvature curves for four-point bending tests.

(a) Experimental setup.

(b) Schematic diagram of the test setup.

Fig. 7. Eccentrically-loaded stub column test configuration.

(a) Test curves for SHS 40×40×2
(Specimens: 1C to 1G).

(b) Test curves for SHS 50×50×2
(Specimens: 2C to 2G).

Fig. 8. Load–end rotation curves for uniaxial bending plus compression tests.

(a) Test curves for specimen SHS 40×40×2-1H ($e_{0y}=19.0$ mm, $e_{0z}=19.0$ mm).

(b) Test curves for specimen SHS 40×40×2-1I ($e_{0y}=13.0$ mm, $e_{0z}=28.0$ mm).

(c) Test curves for specimen SHS 50×50×2-2H ($e_{0y}=14.0$ mm, $e_{0z}=25.0$ mm).

(d) Test curves for specimen SHS 50×50×2-2I ($e_{0y}=14.0$ mm, $e_{0z}=30.0$ mm).

Fig. 9. Load–end rotation curves for biaxial bending plus compression tests.

Fig. 10. Failure mode of specimen SHS 40×40×2-1E under uniaxial eccentric compression.

Fig. 11. Failure mode of specimen SHS 40×40×2-1I under biaxial eccentric compression.

Fig. 12. Experimental and numerical load–end shortening curves for stub column specimen SHS 40×40×2-1A.

Fig. 13. Experimental and numerical normalized moment–curvature curves for beam specimen SHS 40×40×2-1B.

Fig. 14. Experimental and numerical load–end rotation curves for specimen SHS 50×50×2-2F under uniaxial bending plus compression.

Fig. 15. Experimental and numerical load–end rotation curves for specimen SHS 50×50×2-2I under biaxial bending plus compression.

Fig. 16. Experimental and numerical elephant foot failure modes for stub column specimen SHS 40×40×2-1A.

Fig. 17. Experimental and numerical failure modes for beam specimen SHS 40×40×2-1B.

Fig. 18. Experimental and numerical failure modes for specimen SHS 40×40×2-1E subjected to uniaxial bending plus compression.

Fig. 19. Experimental and numerical failure modes for specimen SHS 40×40×2-1I subjected to biaxial bending plus compression.

Fig. 20. Definition of N_u and $N_{u,pred}$ on moment–axial load interaction curve.

(a) Uniaxial bending plus compression.

(b) Biaxial bending plus compression.

Fig. 21. Comparison of test and FE results with the SEI/ASCE-8 and EN 1993-1-4 strength predictions.

Fig. 22. CSM elastic, linear hardening material model.

Fig. 23. Typical CSM linear and nonlinear design interaction curves at the transition slenderness limit.

Fig. 24. Typical simplified CSM linear and bi-linear design interaction curves at the transition slenderness limit.

(a) Uniaxial bending plus compression.

(b) Biaxial bending plus compression.

Fig. 25. Comparison of test and FE results with the SEI/ASCE-8 and simplified CSM strength predictions.

Fig. 26. Comparison of test and FE results with SEI/ASCE-8 and CSM strength predictions.

Fig. 27. Comparison of test and FE results with SEI/ASCE-8 and simplified CSM predictions.

Table 1 Chemical compositions of test cross-sections as stated in the mill certificates.

Cross-section	Grade	C (%)	Si (%)	Mn (%)	P (%)	S (%)	Cr (%)	Ni (%)	N (%)	Mo (%)	Cu (%)	Nb (%)
SHS 40×40×2	1.4509	0.013	0.43	0.22	0.021	0.001	18.26	0.19	0.013	0.02	–	0.38
SHS 50×50×2	1.4509	0.015	0.55	0.20	0.024	0.001	18.27	0.20	0.016	0.02	–	0.36

Table 2 Average measured tensile flat material properties.

Cross-section	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ε_u (%)	ε_f (%)	R-O coefficient		
							n	$n'_{0.2,u}$	$n'_{0.2,1.0}$
SHS 40×40×2	195700	499	–	526	1.2	17.2	6.6	4.2	–
SHS 50×50×2	190100	466	508	515	7.3	24.3	6.6	7.6	7.6

Table 3 Average measured tensile corner material properties.

Cross-section	E (N/mm ²)	$\sigma_{0.2}$ (N/mm ²)	$\sigma_{1.0}$ (N/mm ²)	σ_u (N/mm ²)	ε_u (%)	ε_f (%)	R-O coefficient		
							n	$n'_{0.2,u}$	$n'_{0.2,1.0}$
SHS 40×40×2	200400	639	–	646	0.6	8.1	7.2	–	–
SHS 50×50×2	225800	623	–	658	0.8	8.4	5.3	1.5	–

Table 4 Measured dimensions of stub column specimens.

Cross-section	Specimen ID	L (mm)	H (mm)	B (mm)	t (mm)	r_i (mm)	A (mm ²)	ω_0 (mm)
SHS 40×40×2	1A	150.0	40.0	40.1	2.01	1.8	297.7	0.005
SHS 50×50×2	2A	200.1	50.1	50.2	1.90	2.5	355.8	0.009

Table 5 Summary of stub column test results.

Cross-section	Specimen ID	Ultimate load N_u (kN)	End shortening at ultimate load δ_u (mm)	$N_u/A\sigma_{0.2}$
SHS 40×40×2	1A	183.3	1.85	1.13
SHS 50×50×2	2A	205.0	2.08	1.13

Table 6 Measured dimensions of beam specimens.

Cross-section	Specimen ID	H (mm)	B (mm)	t (mm)	r_i (mm)	ω_0 (mm)
SHS 40×40×2	1B	40.0	40.0	2.00	1.8	0.013
SHS 50×50×2	2B	50.1	50.1	1.90	2.5	0.022

Table 7 Summary of test results for beams.

Cross-section	Specimen ID	Ultimate moment M_u (kNm)	M_u/M_{el}	M_u/M_{pl}	Rotation capacity R
SHS 40×40×2	1B	2.50	1.30	1.09	8.2
SHS 50×50×2	2B	3.25	1.18	1.00	1.6

Table 8 Measured dimensions of uniaxial bending plus compression specimens.

Cross-section	Specimen ID	e_0 (mm)	L (mm)	H (mm)	B (mm)	t (mm)	r_i (mm)	ω_0 (mm)
SHS 40×40×2	1C	8.5	150.0	40.0	40.1	2.02	1.8	0.011
	1D	18.0	150.0	40.1	40.1	2.00	1.8	0.011
	1E	28.4	150.1	40.1	40.0	2.00	1.8	0.011
	1F	36.4	150.0	40.0	40.0	2.00	1.8	0.011
	1G	48.7	150.0	40.0	40.0	2.00	1.8	0.011
SHS 50×50×2	2C	8.8	200.1	50.1	50.1	1.90	2.5	0.015
	2D	20.4	199.9	50.0	50.1	1.91	2.5	0.015
	2E	27.2	200.0	50.0	50.1	1.90	2.5	0.015
	2F	40.5	200.0	50.1	50.2	1.89	2.5	0.015
	2G	50.0	200.0	50.1	50.2	1.90	2.5	0.015

Table 9 Summary of test results for uniaxial bending plus compression specimens.

Cross-section	Specimen ID	e_0 (mm)	N_u (kN)	e' (mm)	M_u (kN m)	ϕ_u (deg)
SHS 40×40×2	1C	8.5	119.0	0.70	1.10	1.65
	1D	18.0	86.6	0.97	1.64	1.98
	1E	28.4	68.9	1.10	2.03	2.49
	1F	36.4	56.2	1.12	2.11	2.49
	1G	48.7	47.4	1.39	2.38	3.06
SHS 50×50×2	2C	8.8	139.2	0.73	1.32	1.38
	2D	20.4	106.2	0.77	2.24	1.45
	2E	27.2	81.6	0.78	2.28	1.51
	2F	40.5	68.9	0.89	2.85	1.55
	2G	50.0	57.9	0.96	2.95	1.75

Table 10 Measured dimensions of biaxial bending plus compression specimens.

Cross-section	Specimen ID	e_{0y} (mm)	e_{0z} (mm)	L (mm)	H (mm)	B (mm)	t (mm)	r_i (mm)	ω_0 (mm)
SHS 40×40×2	1H	19.0	19.0	150.0	40.1	40.0	2.00	1.8	0.011
	1I	13.0	28.0	150.1	40.0	40.0	2.01	1.8	0.011
SHS 50×50×2	2H	14.0	25.0	200.0	50.0	50.2	1.90	2.5	0.015
	2I	14.0	30.0	200.0	50.0	50.1	1.90	2.5	0.015

Table 11 Summary of test results for biaxial bending plus compression specimens.

Cross-section	Specimen ID	e_{0y} (mm)	e_{0z} (mm)	N_u (kN)	e'_y (mm)	e'_z (mm)	M_{uy} (kN m)	M_{uz} (kN m)	ϕ_{uy} (deg)	ϕ_{uz} (deg)
SHS 40×40×2	1H	19.0	19.0	69.9	1.04	1.12	1.40	1.41	1.89	1.94
	1I	13.0	28.0	65.5	0.72	1.12	0.90	1.91	1.22	2.44
SHS 50×50×2	2H	14.0	25.0	86.8	0.62	0.95	1.27	2.25	0.83	1.51
	2I	14.0	30.0	77.9	0.57	1.00	1.13	2.41	0.70	1.53

Table 12 Comparison of the test results with FE results for varying imperfection amplitudes.

Test type	Specimen	Measured amplitude	$t/100$	D & W model
		FE N_u /Test N_u	FE N_u /Test N_u	FE N_u /Test N_u
Concentric stub column tests	SHS 40×40×2-1A	0.93	0.93	0.93
	SHS 50×50×2-2A	0.95	0.94	0.93
4-point bending tests	SHS 40×40×2-1B	0.95	0.95	0.95
	SHS 50×50×2-2B	1.05	1.05	1.05
Uniaxial bending plus compression tests	SHS 40×40×2-1C	0.94	0.94	0.94
	SHS 40×40×2-1D	0.95	0.95	0.95
	SHS 40×40×2-1E	0.93	0.92	0.92
	SHS 40×40×2-1F	0.96	0.96	0.96
	SHS 40×40×2-1G	0.91	0.91	0.91
	SHS 50×50×2-2C	0.97	0.97	0.97
	SHS 50×50×2-2D	0.92	0.91	0.91
	SHS 50×50×2-2E	1.01	1.00	1.00
	SHS 50×50×2-2F	0.97	0.97	0.97
	SHS 50×50×2-2G	0.99	0.99	0.98
Biaxial bending plus compression tests	SHS 40×40×2-1H	0.93	0.93	0.93
	SHS 40×40×2-1I	0.95	0.94	0.95
	SHS 50×50×2-2H	0.96	0.95	0.95
	SHS 50×50×2-2I	0.98	0.97	0.97
Mean		0.97	0.95	0.96
COV		0.03	0.04	0.03

Table 13 Comparison of the uniaxial bending plus compression test and FE results with predicted strengths.

No. of tests: 14 No. of FE simulations: 84	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$	$N_u/N_{u,CSM1}$	$N_u/N_{u,CSM}$
Mean	1.18	1.23	1.13	1.07
COV	11.0%	6.4%	5.6%	4.9%

Table 14 Comparison of the biaxial bending plus compression test and FE results with predicted strengths.

No. of tests: 4 No. of FE simulations: 67	$N_u/N_{u,EC3}$	$N_u/N_{u,ASCE}$	$N_u/N_{u,CSM1}$	$N_u/N_{u,CSM}$
Mean	1.24	1.49	1.15	1.06
COV	18.3%	8.7%	7.7%	6.9%

Table 15 Reliability analysis results calculated according to SEI/ASCE-8.

Method	No. of tests and FE simulations	P_m	V_p	ϕ_0	β_0
CSM	169	1.14	0.072	0.85	3.18
Simplified CSM	169	1.07	0.058	0.85	3.02

Table 16 Reliability analysis results calculated according to EN 1990.

Method	No. of tests and FE simulations	$k_{d,n}$	b	V_δ	V_r	γ_{M0}
CSM	169	3.149	1.093	0.075	0.103	1.05
Simplified CSM	169	3.149	1.056	0.056	0.090	1.04