Behavior and design of normal and high strength steel SHS and RHS columns

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Abstract

The behavior and design of hot-rolled and cold-formed steel square and rectangular hollow section (SHS and RHS) columns, made of both normal and high strength material, are addressed in this paper. A series of experiments on hot-rolled high strength steel SHS columns was first conducted – six tests on S690 SHS 100×100×4 columns and six tests on S770 SHS 120×120×6.3 columns were performed. Finite element (FE) models were developed to replicate the experimental results and to carry out parametric studies to expand the column buckling data pool. The accuracy of the European and North American buckling design rules for normal and high strength steel SHS and RHS columns was evaluated through comparisons with the freshly generated test and FE results, as well as with existing test data collected from a literature. Finally, a modified Eurocode 3 (EC3) approach was proposed and statistically verified in accordance with EN 1990; the new approach features an imperfection factor that is a continuous function of yield strength, reflecting the reducing relative influence of residual stresses and global imperfections with increasing steel grades. Improved consistency in resistance predictions over the existing design provisions is demonstrated across a wide range of steel grades and relative slenderness values.

Keywords: ANSI/AISC 360-16; Column buckling tests; EN 1993-1-1; Experiments; High strength steels; Member stability; Numerical simulation; Rectangular hollow sections (RHS); Square hollow sections (SHS); Testing.

Introduction

Structural steel square and rectangular hollow section (SHS and RHS) elements are widely used in the construction industry as structural components due to their aesthetic appearance and excellent structural qualities, particularly under predominantly compressive loading. SHS and RHS products are commonly manufactured by means of hot-rolling (CEN 2006a), cold-forming (CEN 2006b) or welding four plates into a box shape. With the increasing use of high strength steel SHS and RHS in construction, the development of suitable rules to ensure their safe and efficient design is imperative.

The flexural buckling behavior and design of hot-rolled and cold-formed SHS and RHS columns in normal and high strength steel grades are studied in the present paper. A number of previous experimental investigations into the behavior of structural steel SHS and RHS members under axial compression have been carried out. Laboratory testing on hot-rolled SHS and RHS columns has been conducted by Rondal (1984) and Wang and Gardner (2017), while buckling tests on cold-formed SHS and RHS compression members have been performed and reported by Bjorhovde (1977), Rondal (1984), Key et al. (1988), Sully and Hancock (1996), Pavlovčič et al. (2012), SSAB (2014), Ma (2016) and Somodi and Kovesdi (2017). Several CIDECT projects have also been undertaken to explore the column buckling behavior of SHS and RHS members, including those by Guiaux (1972), Salvarinas et al. (1978), Braham et al. (1979), Sedlacek et al. (1996) and Sedlacek et al. (1999), in which a large number of buckling tests on both hot-rolled and cold-formed columns were reported. Extensive experimental research has also been carried out on welded box section columns by, for example, Rasmussen and Hancock (1995), Ban et al. (2013) and Schillo and Feldmann (2018), but these sections are outside the scope of the present study.

Codified design rules for normal strength steel columns have been developed and used extensively over many decades, whereas corresponding rules for high strength steel columns are still rather limited. The

current version of Eurocode 3 (EC3) includes provisions for the design of columns in steel grades up to S700 (CEN 2005, 2007), while the current version of the American Specification AISC 360 (AISC) is applicable to steel plates with nominal yield strengths up to 690 N/mm², but to hollow sections only up to yield strengths of 485 N/mm² (AISC 2016; ASTM 2015, 2018). In general, the current codified design provisions for high strength steel columns are a simple extension to those for normal strength steels, with scope for improvements in terms of accuracy and consistency.

This study is therefore aimed at expanding the pool of column buckling test data on SHS members in the high strength steel range and developing improved design rules for both normal and high strength steel SHS and RHS members under axial compression. A series of column buckling tests on hot-finished high strength steel SHS is firstly presented. Finite element (FE) models are then established, validated and used to generate supplementary numerical data. The column design rules in Eurocode 3 (EC3) and the American Specification (AISC) are subsequently evaluated using the test and FE data and found to exhibit some shortcomings. A modified EC3 design approach, in which the influence of yield strength is included in the imperfection factor, is then proposed to improve the accuracy and consistency of the resistance predictions; recommendations are also made to improve the AISC column design method. Finally, reliability analysis of the current EC3 design approach and the proposed modified EC3 approach is carried out.

Experimental investigation

General

An experimental program was conducted to investigate the flexural buckling behavior of hot-finished high strength steel SHS members under axial compression. Two quenched and tempered high strength steel SHS profiles in two different grades – S690 SHS 100×100×4 (section ID: 1) and S770 SHS 120×120×6.3 (section ID: 2), were examined. According to the slenderness limits set out in the latest revision to Eurocode 3 – prEN 1993-1-1:2018 (CEN 2018), the studied S690 SHS 100×100×4 and S770

SHS 120×120×6.3 are Class 3/4 and Class 2 cross-sections respectively. In total, twelve SHS column specimens with varying lengths were tested.

Material and cross-section tests

Material tests and stub column tests on the two examined cross-sections have been previously conducted by the authors and reported in Meng and Gardner (2019b); a brief summary of the results is provided herein. The engineering stress-strain curves from the tensile coupon tests are plotted in Fig. 1, and the obtained mechanical properties, including the Young's modulus E , the yield strength f_y , the ultimate strength f_u , the strain at the ultimate strength ε_u and the plastic strain at fracture measured over the standard gauge length ε_f (CEN 2016), are listed in Table 1. Note that these material properties, which were determined from flat tensile coupon tests, are adopted throughout this study to represent the material behavior over the whole cross-section since the uniformity of properties for hot-rolled hollow sections has been previously confirmed by Gardner et al. (2010) and Wang et al. (2017). The key results from the stub column tests are summarized in Table 2, where H , B , R , r and t are the height, breadth, outer corner radius, inner corner radius and thickness of the cross-section respectively, L is the specimen length, $c / t\bar{\varepsilon}$ is the EC3 local slenderness parameter, in which c is the width of the flat region of the cross-section faces, $\bar{\varepsilon}^2 = 235 / f_y$, N_u is the ultimate test load and δ_u is the end shortening at N_u .

Column buckling tests

A total of twelve column specimens, six with S690 SHS 100×100×4 profiles (SC1 series) and six with S770 SHS 120×120×6.3 profiles (SC2 series), were tested to investigate their flexural buckling responses under axial compression. The specimen lengths were varied to cover a wide spectrum of EC3 relative slenderness values $\bar{\lambda}_{EC3}$ (as defined by Eqs. (1) and (2) for Class 1–3 and Class 4 cross-sections respectively, where $N_{c,EC3}$ is the EC3 cross-sectional resistance to compression, A and A_{eff} are the gross and effective cross-sectional areas respectively and N_{cr} is the Euler buckling load) from 0.47 to 1.47. The measured geometric properties and the EC3 relative slenderness for each specimen are summarized in Table 3.

$$\bar{\lambda}_{EC3} = \sqrt{\frac{N_{c,EC3}}{N_{cr}}} = \sqrt{\frac{Af_y}{N_{cr}}} \text{ for Class 1–3 sections} \quad (1)$$

$$\bar{\lambda}_{EC3} = \sqrt{\frac{N_{c,EC3}}{N_{cr}}} = \sqrt{\frac{A_{eff}f_y}{N_{cr}}} \text{ for Class 4 sections} \quad (2)$$

All column buckling tests were conducted using an Instron 2000 kN hydraulic testing machine, as shown in Fig. 2. The column specimens were placed between a pair of wedge plates, with pin-ended boundary conditions achieved through a pair of knife edges at the top and bottom ends. The critical (effective) length L_{cr} of each specimen is equal to the member length L plus an additional length for the knife edge and wedge plate equal to 75 mm at each end, as summarized in Table 3. Adjustable end clamps were employed at both ends to prevent any movement of the specimens during loading. Stainless steel shims were inserted between the specimen ends and the wedge plates when necessary to eliminate any small gaps arising from the out-of-flatness of the end sections. Four electrical resistance strain gauges were affixed at the mid-height of the two opposite faces of the test specimens parallel to the axis of buckling at a distance of $3t$ from the edges, for the measurement of the maximum and minimum outer fiber strains. The lateral deflection at mid-height and rotations at the top and bottom ends of the columns were measured using a linear variable displacement transducer (LVDT) and a pair of inclinometers respectively. The test outputs, including the load, machine displacement and readings from the strain gauges, LVDT and inclinometers, were recorded at 0.5-s intervals using the data acquisition equipment DATASCAN and logged using the DSLOG software.

The global imperfections ω_g of the columns consisted of the out-of-straightness of the member and the initial loading eccentricity, the latter of which was adjusted such that a total imperfection (out-of-straightness + initial loading eccentricity) as close as possible to $L_{cr}/1000$ was achieved for each test; this value is the typically assumed global imperfection amplitude in the development of column buckling design rules (Beer and Schulz 1970; Ziemian 2010). Prior to testing, each specimen was preloaded to about 10% of the predicted failure load, enabling the global imperfection amplitude ω_g to be back-calculated using Eq. (3), where I is the second moment of area about the axis of buckling, ε_{max} and ε_{min} are the average strains (from the pair of strain gauges) on the concave and convex sides of each

column respectively, N is the applied axial load and Δ is the measured mid-height lateral deflection. The initial loading eccentricity was then carefully adjusted, and the procedure repeated, until the resulting ω_g was sufficiently close to the target value of $L_{cr}/1000$. The final values of ω_g for all column tests are reported in Table 3.

$$\omega_g = \frac{EI(\varepsilon_{\max} - \varepsilon_{\min})}{HN} - \Delta \quad (3)$$

The load-mid-height lateral deflection curves obtained from the tests are presented in Figs. 3 and 4 for the S690 SHS 100×100×4 and S770 SHS 120×120×6.3 columns respectively. The ultimate loads N_u are reported in Table 3, and all deformed test specimens are displayed in Figs. 5 and 6. All tested specimens buckled in the direction dictated by the global imperfections. For the shorter three specimens in the SC1 series, i.e. SC1-1, SC1-2 and SC1-3, failure was dominated by local buckling, while for the longer specimens – SC1-4, SC1-5 and SC1-6, only global buckling was evident at the ultimate load, with local buckling occurring after the peak load under the combination of a lower level of axial compression but an increasing second-order bending moment. The local buckling occurred in an abrupt manner for the SC1 series, resulting in a sudden drop in applied load and a jump in the recorded deformations, as indicated by the dashed lines in Fig. 3. For the SC2 series, all specimens failed by global buckling, with local buckling forming gradually beyond the ultimate load, except for specimens SC2-5 and SC2-6, where the tests were terminated before the onset of local buckling.

Numerical simulations

General

In addition to the laboratory testing, a numerical modeling program was carried out to simulate the buckling behavior of SHS and RHS members under axial compression. Finite element (FE) models were developed and validated against available column test results on the hot-rolled and cold-formed SHS and RHS. Additional numerical results for a range of steel grades and geometries were then generated from parametric studies as an expansion of the data pool for SHS and RHS column buckling.

The flexural buckling behavior of SHS and RHS members was simulated by means of geometrically and materially nonlinear analyses with imperfections (GMNIA) using the finite element analysis package *Abaqus*. The basic assumptions of the numerical modeling approach are described herein. The four-noded doubly curved shell element with reduced integration S4R (*Abaqus*) was employed throughout the investigation, as commonly used in previous numerical studies on structural hollow sections, such as those reported by Yun and Gardner (2018), Meng et al. (2019) and Zhao et al. (2019). In the flat regions of the modeled cross-sections, a finer mesh of $(H+B)/30$ was applied within a length of $4H$ of the mid-height of the columns to capture the potential local buckling, while a coarser mesh of $(H+B)/15$ was adopted for the remainder of the member length to reduce computational time; for each corner region, six elements were employed. Symmetry was also exploited by establishing quarter-models with suitable symmetry boundary conditions applied (see Fig. 7) to further enhance the computational efficiency. The end section of the models was coupled to a reference point through kinematic coupling, with boundary conditions applied to the reference point to reflect the corresponding test end conditions.

Both local and global geometric imperfections were introduced into the FE models in the form of the lowest local and global elastic buckling shapes from a prior linear bifurcation analysis (LBA) performed for each member. Residual stresses were not modeled explicitly for either the hot-rolled or the cold-formed SHS and RHS members. This is because for hot-rolled hollow sections, residual stress magnitudes have been shown to be negligible compared with the yield strength (Law and Gardner 2012; Wang et al. 2016), while for cold-formed hollow sections, the effect of the dominant bending residual stresses is already implicitly incorporated into the stress-strain curves obtained from tensile tests performed on coupons cut from the examined profiles (Rasmussen and Hancock 1993; Jandera et al. 2008). It has also been confirmed in a number of previous numerical studies, such as Wang and Gardner (2017), Huang and Young (2018), Meng and Gardner (2019a) and Toffolon and Taras (2019), that FE models without the explicit incorporation of residual stresses are capable of accurately replicating the experimentally observed physical responses of both hot-rolled and cold-formed steel tubular members.

To confirm the accuracy of the developed FE models, the numerical predictions were compared with the key results from experiments, including the ultimate loads, load-deformation histories and failure modes. The twelve column buckling tests on hot-finished SHS from this study and eleven tests on cold-formed SHS and RHS from a previous study by Ma (2016) were adopted for the validation study. A local imperfection amplitude of $\omega_l = c / 200$ was assumed, as recommended in EN 1993-1-5:2006 (CEN 2006c) and successfully employed in several previous numerical studies e.g. Yun et al. (2018) and Walport et al. (2019). Note that the measured local imperfection amplitudes (Ma et al. 2016; Meng and Gardner 2019b) for the considered SHS and RHS profiles were, on average, slightly smaller than the value of $c / 200$ recommended in EN 1993-1-5:2006 (CEN, 2006c), and thus, slightly conservative resistance predictions from the FE models were anticipated. A series of global imperfection amplitudes ω_g – the measured values, $L_{cr} / 500$, $L_{cr} / 1000$ and $L_{cr} / 2000$, was incorporated into the FE models to assess their sensitivity to variation in global imperfection levels. For the hot-rolled sections, the measured material properties were applied uniformly to the full modeled cross-sections, whereas for the cold-formed sections, corner strength enhancements that arise due to plastic deformation during the rolling process were incorporated by assigning the measured corner material properties to the corner regions of the cross-sections plus to an extension of $2t$ into the flat regions (Cruise and Gardner 2008; Afshan et al. 2013). Deviation of the actual distribution of mechanical properties from this idealized pattern is a potential source of modeling error, though accurate results have been consistently obtained in previous numerical models of cold-formed SHS and RHS developed on this basis (Yun and Gardner 2018; Ma et al. 2019). Comparisons of the ultimate loads $N_{u,FE}$ derived from the FE models with those obtained from the experiments $N_{u,test}$ are shown in Table 4. The FE models can be seen to be relatively sensitive to the global imperfection amplitude; when the measured values of ω_g were utilized, excellent accuracy and consistency in the FE predictions were achieved, while with ω_g taken as $L_{cr} / 1000$, the predictions were accurate, but slightly conservative. Excellent agreement between the test and numerical load-deformation histories and failure modes was also observed, as shown in Figs. 8 and 9.

Overall, it can be concluded that the developed FE models are capable of accurately replicating the buckling behavior of hot-rolled and cold-formed SHS and RHS members under axial compression.

Parametric studies

Following the validation study, the developed FE models were used in parametric studies for the generation of additional column buckling data on hot-rolled and cold-formed SHS and RHS over a wider range of steel grades, cross-section geometries and member lengths. Five steel grades from normal to high strengths – S355, S460, S500 and S690/S700 and S900, were considered for both hot-rolled and cold-formed sections. The material models developed by Yun and Gardner (2017) and Gardner and Yun (2018) were employed for the description of the respective stress-strain curves; the adopted input parameters are listed in Table 5. The nominal values of f_y and f_u from prEN 1993-1-1:2018 (CEN 2018) were used for steel grades up to S700, while for S900, f_u was taken as $1.05f_y$ according to the ductility requirements given in EN 1993-1-12:2007 (CEN 2007). Note that for the cold-formed SHS and RHS, the predictive models for corner strength enhancements by Rossi et al. (2013) were used in conjunction with the stress-strain models of Gardner and Yun (2018), and the resulting corner stress-strain relationships were assigned to the corner regions of the cross-section plus to an extension of $2t$ into the adjacent flat regions. Two sets of cross-section outer dimensions – 100×100 and 100×50 ($H \times B$, in mm), were examined, with the thickness t varied to achieve a spectrum of local slenderness within the EC3 Class 1–3 domain. The modeled member lengths were varied from 160 mm to 6000 mm, leading to a range of global slenderness values $\bar{\lambda}$ from 0.16 to 2.62.

A total of 2300 numerical results on hot-rolled and cold-formed SHS and RHS columns, covering normal to high strength steel members, were generated. These numerical data are combined with existing experimental data and used in the next section for the assessment of existing and development of new design provisions for hot-rolled and cold-formed steel SHS and RHS columns.

Stability design of SHS and RHS columns

General

The numerical results generated in the previous section have been supplemented with experimental results collected from the literature. In total, 588 existing test data have been collected, with 314 falling into the Class 1–3 range in accordance with prEN 1993-1-1:2018 (CEN 2018). The freshly generated numerical column buckling data, combined with the collected test results for members with nonslender cross-sections, are plotted against the codified column buckling curves in Figs. 10. It should be noted that a relatively high level of scatter is observed in the test data points, which is attributed largely to the sensitivity of the column buckling resistance to the inherently scattered global imperfections and initial loading eccentricities, as well as variations in the test conditions.

An assessment of the accuracy of the codified column buckling design provisions in prEN 1993-1-1:2018 (CEN 2018) and ANSI/AISC 360-16 (AISC 360-16) for hot-rolled and cold-formed SHS and RHS with varying steel grades is firstly conducted using the established test and FE database. To address the shortcomings revealed in both design codes, a modified EC3 design approach is developed, in which the imperfection factor is defined as a continuous function of yield strength, leading to improved accuracy and consistency; recommendations for improvements to the AISC column buckling design approach are also made. Reliability analysis of the current EC3 and proposed modified EC3 column design approaches is then conducted to determine a suitable partial safety factor.

Background of Ayrton-Perry formulation

The background to the Ayrton-Perry formulation (Ayrton and Perry 1886) to describe the buckling behavior of imperfect columns is first presented. The derivation is made based on the following assumptions: (a) a simply-supported prismatic member under axial compression; (b) an initial geometric imperfection in the form of a half sine wave with a maximum value e_0 at mid-height; (c) a nonslender cross-section and (d) a first-yield failure criterion. For a member under an axial force N , the maximum total lateral deflection at mid-height $v_{\max}$ (equal to the initial geometric imperfection amplitude e_0 plus

the additional mid-height lateral deflection Δ) is given by Eq. (4), where N_{cr} is the Euler buckling load, as given by Eq. (5). The load corresponding to first-yield of the member $N_{b,R}$ can be expressed by Eq. (6), where A is the cross-sectional area and W_{el} is the elastic section modulus.

$$v_{\max} = \frac{1}{1 - N / N_{cr}} e_0 \quad (4)$$

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} \quad (5)$$

$$\frac{N_{b,R}}{A} + \frac{1}{1 - N_{b,R} / N_{cr}} \frac{N_{b,R} e_0}{W_{el}} = f_y \quad (6)$$

By substituting the relative slenderness $\bar{\lambda}$ (as defined by Eq. (7)) and the column buckling reduction factor χ (as defined by Eq. (8), where $N_{b,R}$ is the column buckling resistance based on the assumption of failure at the point of first yield) into Eq. (6), Eq. (9) is determined (Boissonnade et al. 2006), where η is the generalized initial imperfection factor, as given by Eq. (10).

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \quad (7)$$

$$\chi = \frac{N_{b,R}}{Af_y} \quad (8)$$

$$(1 - \chi) \left(1 - \chi \bar{\lambda}^2 \right) = e_0 \frac{A}{W_{el}} \chi = \eta \chi \quad (9)$$

$$\eta = e_0 \frac{A}{W_{el}} \quad (10)$$

Finally, the column buckling reduction factor χ can be derived from Eq. (9), as given by Eqs. (11) and (12), with the theoretical column buckling resistance being $N_{b,R} = \chi Af_y$.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad (11)$$

$$\Phi = 0.5 \left(1 + \eta + \bar{\lambda}^2 \right) \quad (12)$$

270 The EC3 column buckling design curves originate from the Ayrton-Perry formulation and have been
 271 calibrated against experimental and numerical data for different types of cross-section (Beer and Schulz
 272 1970; Starting and Vos 1973; Sfintesco and Carpena 1977; Chan et al. 2010; Taras and Greiner 2010).

273 The EC3 relative slenderness $\bar{\lambda}_{EC3}$ is defined by Eqs. (1) and (2) for Class 1–3 and Class 4 sections
 274 respectively, where A_{eff} is the effective area of the cross-section, allowing for local buckling and
 275 determined in accordance with EN 1993-1-5:2006 (CEN 2006c).

276 The EC3 column buckling reduction factor χ_{EC3} is determined from the modified Ayrton-Perry formula,
 277 as given by Eqs. (13) to (15), where the EC3 generalized initial imperfection factor η_{EC3} given by Eq.
 278 (14) is adopted to account for the combined effects of initial out-of-straightness and residual stresses
 279 and α is the imperfection factor, as tabulated in Table 6 for different production routes and steel grades.

$$280 \quad \chi_{EC3} = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}_{EC3}^2}} \quad \text{but } \chi_{EC3} \leq 1.0 \quad (13)$$

$$281 \quad \eta_{EC3} = \alpha (\bar{\lambda}_{EC3} - 0.2) \quad (14)$$

$$282 \quad \Phi = 0.5 \left(1 + \eta_{EC3} + \bar{\lambda}_{EC3}^2 \right) = 0.5 \left(1 + \alpha (\bar{\lambda}_{EC3} - 0.2) + \bar{\lambda}_{EC3}^2 \right) \quad (15)$$

283 Finally, the design column buckling resistance $N_{b,Rd}$, as expressed by Eq. (16), is calculated as the
 284 product of the EC3 column buckling reduction factor χ_{EC3} and the EC3 cross-section resistance $N_{c,EC3}$,
 285 to give $N_{b,EC3} = \chi_{EC3} N_{c,EC3}$, divided by the partial safety factor γ_{M1} , which has a recommended value of
 286 1.0 in prEN 1993-1-1:2018 (CEN 2018).

$$287 \quad N_{b,Rd} = \frac{N_{b,EC3}}{\gamma_{M1}} = \frac{\chi_{EC3} N_{c,EC3}}{\gamma_{M1}} \quad (16)$$

288 To assess the accuracy of the EC3 column buckling design provisions for SHS and RHS members, the
 289 ultimate loads from the column buckling tests and FE simulations N_u are normalized by the unfactored
 290 resistance predictions from EC3 $N_{b,EC3}$ and plotted against the relative slenderness $\bar{\lambda}$ in Fig. 11. Table
 291 7 summarizes the mean values and coefficients of variation (COV) of the ratio $N_u / N_{b,EC3}$. Note that the

yield strength f_y for the cold-formed SHS and RHS is taken as the weighted average yield strength incorporating the strength enhancements in the corner region, as given by Eq. (17), where $f_{y,f}$ and $f_{y,c}$ are the yield strengths of the flat and corner regions respectively, and A_f and A_c are the total areas of the flat and corner regions respectively.

$$f_y = \frac{f_{y,f}A_f + f_{y,c}A_c}{A} \text{ for cold-formed SHS and RHS} \quad (17)$$

For the hot-rolled SHS and RHS, the EC3 design approach generally yields accurate column buckling resistance predictions, though a relatively wide scatter band is observed since the influence of the yield strength on the column buckling behavior is not fully captured. For the cold-formed SHS and RHS, the EC3 predictions generally fall on the safe side, but exhibit an increasing level of conservatism with increasing yield strength, owing again to the influence of yield strength on the buckling behavior not being fully captured. Furthermore, a number of data points appear on the unsafe side for $\bar{\lambda} < 0.7$ for both hot-rolled and cold-formed SHS and RHS; this stems, to a large extent, from the fixed plateau length of $\bar{\lambda}_{EC3,0} = 0.2$ that features in the current EC3 column buckling curves.

Modified EC3 approach

Residual stresses and global imperfections play an important role in the buckling behavior of compressed steel members. Residual stresses have been shown to be approximately constant regardless of the steel grade (ECCS 1984), and hence reduce as a proportion of the yield strength for higher grades. In terms of global imperfections, the original Ayrton-Perry formulation can be utilized to examine their influence on the buckling strength of columns in different grades. Considering a pin-ended column with an initial bow imperfection of amplitude $e_0 = L / 1000$ and no residual stress, the generalized initial imperfection factor η can be rearranged into Eq. (18), indicating that η is proportional to the product of the relative slenderness $\bar{\lambda}$ and $f_y^{-1/2}$ (or, equivalently to the product of $\bar{\lambda}$ and $\varepsilon = \sqrt{235/f_y}$) for a given cross-section profile. This means that as the yield strength increases, the generalized initial imperfection factor becomes smaller for a given level of relative slenderness, corresponding to a less

detrimental effect from global imperfections on the column buckling curve. Overall, therefore, a higher normalized column buckling resistance is anticipated with increasing yield strength.

$$\eta = \frac{L}{1000} \frac{A}{W_{el}} = \left(\frac{\pi}{1000} \frac{\sqrt{EIA}}{W_{el}} \right) \frac{\bar{\lambda}}{\sqrt{f_y}} = \left(\frac{\pi}{1000\sqrt{235}} \frac{\sqrt{EIA}}{W_{el}} \right) \bar{\lambda} \varepsilon \quad (18)$$

However, in the current EC3 design approach, these beneficial effects are only partially captured for hot-rolled SHS and RHS in a step wise manner by applying different imperfection factors α for steel grades above and below S460, and are ignored for cold-formed SHS and RHS. In light of this, a modified imperfection factor α_{mod} , incorporating ε , is proposed to take due account of the influence of yield strength on the column buckling behavior in a consistent and continuous manner, as given by Eqs. (19) and (20) for hot-rolled and cold-formed SHS and RHS respectively.

$$\alpha_{mod} = 0.24\varepsilon \text{ for hot-rolled SHS and RHS} \quad (19)$$

$$\alpha_{mod} = 0.56\varepsilon \text{ for cold-formed SHS and RHS} \quad (20)$$

On the other hand, to address the optimistic EC3 buckling strength predictions in the lower slenderness range, a shorter plateau length of $\bar{\lambda}_{mod-EC3,0} = 0.1$ is proposed to supersede the current EC3 plateau length of $\bar{\lambda}_{EC3,0} = 0.2$, resulting in slightly lower resistance predictions for stockier columns. The modified EC3 generalized initial imperfection factor $\eta_{mod-EC3}$, incorporating both the proposed modified imperfection factor and shortened plateau is expressed by Eq. (21).

$$\eta_{mod-EC3} = \alpha_{mod} (\bar{\lambda}_{EC3} - 0.1) \quad (21)$$

Finally, the modified EC3 column buckling reduction factor $\chi_{mod-EC3}$ is given by Eqs. (22) and (23).

$$\chi_{mod-EC3} = \frac{1}{\Phi_{mod} + \sqrt{\Phi_{mod}^2 - \bar{\lambda}_{EC3}^2}} \text{ but } \chi_{mod-EC3} \leq 1.0 \quad (22)$$

$$\Phi_{mod} = 0.5 \left(1 + \eta_{mod-EC3} + \bar{\lambda}_{EC3}^2 \right) = 0.5 \left(1 + \alpha_{mod} (\bar{\lambda}_{EC3} - 0.1) + \bar{\lambda}_{EC3}^2 \right) \quad (23)$$

The accuracy of the proposed modified EC3 design approach for hot-rolled and cold-formed SHS and RHS columns is now assessed using the obtained experimental and numerical data. Comparisons of the

test and numerical results N_u with the predictions from the proposed approach $N_{b,mod-EC3}$ are presented in a normalized manner in Fig. 12, with the statistical results summarized in Table 7. The data points associated with the various steel grades now follow a significantly tighter trend for both hot-rolled and cold-formed SHS and RHS columns, showing that the influence of yield strength on the column buckling behavior is successfully captured by the modified imperfection factor α_{mod} . Meanwhile, owing to the adoption of a shorter plateau length, an improved level of accuracy over the EC3 approach is achieved in the lower slenderness range. The improvements in accuracy and consistency over the current EC3 approach are further revealed quantitatively by the statistical results presented in Table 7, particularly for the cold-formed SHS and RHS columns.

American Specification – ANSI/AISC 360-16

The design rules for SHS and RHS compression members in the American Specification ANSI/AISC 360-16 (AISC 2016) are considered in this section, with the notation adjusted for consistency with EC3. Unlike the EC3 relative slenderness that features the effective cross-sectional area for Class 4 cross-sections, the AISC relative slenderness $\bar{\lambda}_{AISC}$ is defined based on the gross cross-section properties regardless of the local slenderness, as given by Eq. (24).

$$\bar{\lambda}_{AISC} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i} \frac{1}{\pi} \sqrt{\frac{f_y}{E}} \quad (24)$$

The AISC column buckling reduction factor χ_{AISC} is determined from a single two-stage column buckling curve for all production routes and steel grades, as described by Eqs. (25) and (26).

$$\chi_{AISC} = 0.658^{\bar{\lambda}_{AISC}^2} \quad \text{for } \bar{\lambda}_{AISC} \leq 1.5 \quad (25)$$

$$\chi_{AISC} = \frac{0.877}{\bar{\lambda}_{AISC}^2} \quad \text{for } \bar{\lambda}_{AISC} > 1.5 \quad (26)$$

The design buckling resistance under axial compression $N_{b,Rd}$ can then be derived from Eqs. (27) and (28) for nonslender and slender SHS and RHS respectively, where $\phi_c = 0.9$ is the resistance factor for compression and A_{eff} is the effective cross-sectional area calculated in accordance with ANSI/AISC 360-16 (AISC 2016).

$$N_{b,Rd} = \phi_c N_{b,AISC} = \phi_c \chi_{AISC} f_y A \text{ for nonslender SHS and RHS} \quad (27)$$

$$N_{b,Rd} = \phi_c N_{b,AISC} = \phi_c \chi_{AISC} f_y A_{eff} \text{ for slender SHS and RHS} \quad (28)$$

The accuracy of the AISC column buckling design curve is now evaluated through comparisons of the test and FE data N_u with the unfactored AISC resistance predictions $N_{b,AISC}$, as shown in Fig. 13 and Table 7. The AISC buckling curve generally provides a lower bound to the test and FE data points for the hot-rolled SHS and RHS columns, whereas for the cold-formed compression members, a large number of data points appear on the unsafe side. In general, the shape of the AISC column buckling curve matches less well with the trend of the column buckling data compared to the Ayrton-Perry formulation used in EC3, leading to relatively high values of COV in Table 7 for both hot-rolled and cold-formed SHS and RHS columns. Furthermore, the influence of yield strength on the column buckling curve is not considered in the AISC approach, leading to reduced accuracy and increased scatter when high strength steel grades are considered. The following recommendations are therefore made herein for improvements to the AISC column design approach: (a) different buckling curves should be used for hot-rolled and cold-formed columns to account for their distinct buckling responses arising from the different stress-strain characteristics and residual stress distributions and (b) the influence of yield strength on the column buckling curve should be duly incorporated.

Reliability analysis

The reliability of the buckling design rules in prEN 1993-1-1:2018 (CEN 2018) and the proposed modified EC3 approach for normal and high strength steel SHS and RHS columns is evaluated in this section following the standard procedure set out in EN 1990:2002 (CEN 2002). As provided in Annex E of prEN 1993-1-1:2018 (CEN 2018), the material over-strength $f_{y,m} / f_{y,n}$ (i.e. the ratio between the mean and nominal yield strengths) is taken as 1.2, 1.15 and 1.1 for steel grades below S460, equal to S460 and above S460 respectively, with the corresponding COV of yield strength V_{fy} equal to 0.05, 0.04 and 0.035 respectively. The COV of the Young's modulus V_E is taken equal to 0.03 (CEN 2018), while the COV of the cross-sectional area V_A is taken as 0.026 for SHS and RHS, as calculated based on the statistical parameters of the geometric dimensions provided in Annex E of prEN 1993-1-1:2018

(CEN 2018) and following the procedure described in Afshan et al. (2015). The mean values of the Young's modulus E and all the geometric properties are taken equal to the nominal values except for the thickness t , which is taken equal to 0.99 times the nominal thickness of the SHS and RHS according to Annex E of prEN 1993-1-1:2018 (CEN 2018).

The column buckling design function can be expressed in terms of the Young's modulus E , yield strength f_y and the cross-sectional area A , as given by Eq. (29), where the dependency of the resistance function on the variables (i.e. the values of the exponents c , d and e) can be derived for each individual test or numerical simulation following the method set out in Afshan et al. (2015). The combined coefficient of variation of the material and geometric basic variables V_{rt} , incorporating the varying degrees of influence from the variability of f_y , A and E , can then be determined from Eq. (30) for each test or simulation.

$$N_{b,R} = k f_y^c A^d E^e \quad (29)$$

$$V_{rt} = \sqrt{(cV_{f_y})^2 + (dV_A)^2 + (eV_E)^2} \quad (30)$$

The design fractile factor $k_{d,n}$, the COV of the test and FE resistances relative to the predictions from the resistance model V_{δ} and the combined COV incorporating the variability of the resistance model and the basic variables V_r were calculated in accordance with EN 1990:2002 (CEN 2002). The correction factor b was calculated using Eq. (31) instead of the least squares method recommended in EN 1990:2002 (CEN 2002), where n is the number of test and FE data, r_e is the experimental resistance and r_t is the theoretical resistance determined from the resistance model. This approach prevents the resulting value of b from being biased towards the test and FE specimens with higher resistance values (Meng et al. 2019).

$$b = \frac{1}{n} \sum_{i=1}^n \frac{r_{e,i}}{r_{t,i}} \quad (31)$$

The test and FE data were divided by relative slenderness into three groups: $\bar{\lambda} \leq 0.75$, $0.75 < \bar{\lambda} \leq 1.25$ and $1.25 < \bar{\lambda} \leq 2$; those with $\bar{\lambda} > 2$ are deemed beyond the practical range and were therefore not

considered in the reliability analysis. The key reliability analysis results are reported in Table 8, where γ_{M1}^* is the required value of partial safety factor for member buckling design. Note that the values of b , and V_δ and V_t shown in Table 8 were obtained based on the entire set of data for illustration purposes. The required partial safety factors for the proposed approach are equal to 1.063 and 1.041 for hot-rolled and cold-formed SHS and RHS respectively, which are slightly higher than the target value of 1.0, but lower than the corresponding values of 1.086 and 1.054 for the current EC3 approach. Therefore, the EC3 recommended partial safety factor γ_{M1} for member buckling design is deemed applicable to the proposed modified EC3 approach, particularly in light of the reliability recommendations given in Simões da Silva et al. (2017).

Conclusions

A comprehensive study into the buckling behavior of structural steel SHS and RHS columns has been presented in this paper. A total of twelve hot-finished S690 and S770 SHS columns, covering a wide spectrum of relative slenderness values from 0.47 to 1.47, were tested with pin-ended boundary conditions. Finite element models were then established and validated against the test results generated herein on hot-finished SHS columns and existing test results on cold-formed SHS and RHS columns; excellent agreements between the test and numerical responses were achieved. Subsequent parametric studies provided over 2000 column buckling resistance data over a wider range of steel grades and cross-section and member geometries. The generated experimental and numerical data, combined with further existing test results collected from the literature, were utilized to assess current codified design rules for hot-rolled and cold-formed SHS and RHS columns. The Ayrton-Perry based formula specified in prEN 1993-1-1:2018 was shown to provide accurate buckling strength predictions for both hot-rolled and cold-formed SHS and RHS columns, though with some scope for improvement. A modified EC3 approach, featuring a modified imperfection factor linked to the yield strength of the material and a shortened plateau length, has been developed and shown to provide more consistent resistance predictions than the current EC3 provisions. The column design rules in the American Specification

AISC 360-16 were also examined; the use of different column buckling curves for hot-rolled and cold-formed sections and for account to be taken of the improved normalized response of columns with increasing yield strength was proposed. Finally, reliability analysis was conducted for the current EC3 and the proposed modified EC3 column buckling design approaches in accordance with EN 1990:2002, confirming that the EC3 recommended partial safety factor of unity is suitable for use with the proposed approach.

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Fig. 1. Engineering stress-strain curves from tensile coupon tests (Meng and Gardner, 2019b)

Fig. 2. Column buckling test setup

Fig. 3. Experimental load-mid-height lateral deflection curves for S690 SHS 100×100×4 (SC1 series)

Fig. 4. Experimental load-mid-height lateral deflection curves for S770 SHS 120×120×6.3 (SC2 series)

Fig. 5. Failure modes of S690 SHS 100×100×4 (SC1 series, scale in mm)

Fig. 6. Failure modes of S770 SHS 120×120×6.3 (SC2 series, scale in mm)

Fig. 7. Illustration of quarter-model for SHS and RHS columns

Fig. 8. Comparisons of typical load-deformation curves from column buckling tests and FE models (SC1-5 and SC2-3)

Fig. 9. Comparisons of typical failure modes from column buckling tests and FE models (SC1-5)

(a) Hot-rolled SHS and RHS

(b) Cold-formed SHS and RHS

Fig. 10. Comparisons of column buckling test and FE data with codified column buckling curves for (a) hot-rolled SHS and RHS and (b) cold-formed SHS and RHS

(a) Hot-rolled SHS and RHS

(b) Cold-formed SHS and RHS

Fig. 11. Comparisons of column buckling test and FE resistances with unfactored resistance predictions from prEN 1993-1-1:2018 for (a) hot-rolled SHS and RHS and (b) cold-formed SHS and RHS

(a) Hot-rolled SHS and RHS

(b) Cold-formed SHS and RHS

Fig. 12. Comparisons of column buckling test and FE resistances with unfactored resistance predictions from proposed modified EC3 approach for (a) hot-rolled SHS and RHS and (b) cold-formed SHS and RHS

(a) Hot-rolled SHS and RHS

(b) Cold-formed SHS and RHS

Fig. 13. Comparisons of column buckling test and FE resistances with unfactored resistance predictions from ANSI/AISC 360-16 for (a) hot-rolled SHS and RHS and (b) cold-formed SHS and RHS

Table 1. Average measured material properties from tensile coupon tests (Meng and Gardner 2019b)

Cross-section	E	f_y	f_u	ε_u	ε_f	f_u / f_y	$\varepsilon_u / \varepsilon_y$
-	N/mm ²	N/mm ²	N/mm ²	%	%	-	-
S690 SHS 100×100×4	211000	787.3	825.8	6.2	17.1	1.049	16.6
S770 SHS 120×120×6.3	206300	859.0	888.9	5.7	15.4	1.035	13.8

Table 2. Key results from stub column tests (Meng and Gardner 2019b)

Cross-section	H	B	R	r	t	L	$c / t\varepsilon$	N_u	δ_u
-	mm	mm	mm	mm	mm	mm	-	kN	Mm
S690 SHS 100×100×4	100.76	100.05	7.9	4.4	3.99	300.03	38.5	1184.3	1.19
S770 SHS 120×120×6.3	119.00	118.62	9.5	3.8	6.29	360.20	30.0	2492.8	3.38

Table 3. Measured geometric properties of test specimens and key results from column buckling tests

Cross-section	ID	H	B	R	r	t	L	L_{cr}	$\bar{\lambda}_{EC3}$	ω_g	N_u
-	-	mm	mm	mm	mm	mm	mm	mm	-	mm	kN
S690 SHS 100×100×4	SC1-1	100.49	100.48	8.4	4.3	4.04	802.0	952.0	0.47	0.98	1148.1
	SC1-2	100.13	100.94	7.9	4.3	4.01	1199.0	1349.0	0.67	1.37	1053.6
	SC1-3	100.17	100.70	8.6	4.5	4.02	1599.8	1749.8	0.87	1.77	1071.1
	SC1-4	99.53	101.34	7.8	4.6	4.02	2000.3	2150.3	1.08	2.06	807.0
	SC1-5	100.98	100.24	8.4	4.6	4.03	2397.4	2547.4	1.26	2.51	668.4
	SC1-6	100.40	100.47	8.3	4.7	4.02	2800.0	2950.0	1.47	2.95	498.1
S770 SHS 120×120×6.3	SC2-1	120.12	118.66	10.5	4.6	6.37	2003.0	2153.0	0.96	2.22	1873.5
	SC2-2	118.71	120.35	11.3	4.6	6.37	2201.3	2351.3	1.06	2.29	1610.6
	SC2-3	120.13	119.18	11.3	5.3	6.34	2399.8	2549.8	1.14	2.46	1549.2
	SC2-4	119.30	119.13	10.0	4.2	6.30	2601.8	2751.8	1.23	2.74	1343.3
	SC2-5	119.07	119.17	9.5	3.6	6.33	2798.9	2948.9	1.32	2.72	1223.8
	SC2-6	119.32	119.23	9.6	3.7	6.31	2923.0	3073.0	1.38	2.96	1148.4

Table 4. Comparisons between column buckling test results and FE predictions with varying levels of global imperfections

Source of data	Production route	Evaluation parameter	Measured ω_g	$L_{cr} / 500$	$L_{cr} / 1000$	$L_{cr} / 2000$
			$N_{u,FE} / N_{u,test}$	$N_{u,FE} / N_{u,test}$	$N_{u,FE} / N_{u,test}$	$N_{u,FE} / N_{u,test}$
-	-	-	-	-	-	-
This study (12 tests)	Hot-rolled	Mean	0.976	0.923	0.975	1.025
		COV	0.033	0.045	0.033	0.045
Ma (2016) (11 tests)	Cold-formed	Mean	0.981	0.928	0.955	0.960
		COV	0.040	0.071	0.061	0.050

Table 5. Key material properties used in parametric studies

Grade	Hot-rolled SHS and RHS			Cold-formed SHS and RHS		
	E N/mm ²	f_y N/mm ²	f_u N/mm ²	E N/mm ²	f_y N/mm ²	f_u N/mm ²
S355	210000	355	490	210000	355	430
S460	210000	460	540	210000	460	520
S550	210000	550	600	210000	550	600
S690/S700	210000	690	770	210000	700	750
S900	210000	900	945	210000	900	945

Table 6. Values of imperfection factor α for hollow sections in prEN 1993-1-1:2018 (CEN 2018)

Cross-section type	α (S235–S420)	α (S460–S700)
-	-	-
Hot-rolled hollow sections	0.21 (curve a)	0.13 (curve a ₀)
Cold-formed hollow sections	0.49 (curve c)	0.49 (curve c)

Table 7. Comparisons of column buckling test and FE results with predicted resistances for hot-rolled and cold-formed SHS and RHS

Cross-section type	Evaluation parameter	prEN1993-1-1:2018	Modified EC3	ANSI/AISC 360-16
		$N_u / N_{b,EC3}$	$N_u / N_{b,mod-EC3}$	$N_u / N_{b,AISC}$
-	-	-	-	-
Hot-rolled SHS and RHS	Mean	1.029	1.053	1.074
(96 tests + 1150 simulations)	COV	0.037	0.040	0.052
Cold-formed SHS and RHS	Mean	1.116	1.084	0.951
(108 tests + 1150 simulations)	COV	0.064	0.051	0.122

Table 8. Reliability analysis results for (a) prEN 1993-1-1:2018 and (b) modified EC3 approach

(a) prEN 1993-1-1:2018

Cross-section type	n	$k_{d,n}$	b	V_δ	V_r	γ_{M1}^*
-	-	-	-	-	-	-
Hot-rolled SHS and RHS	1123	3.098	1.030	0.038	0.064	1.086
Cold-formed SHS and RHS	1128	3.098	1.115	0.068	0.084	1.054

(b) Modified EC3 approach

Cross-section type	n	$k_{d,n}$	b	V_δ	V_r	γ_{M1}^*
-	-	-	-	-	-	-
Hot-rolled SHS and RHS	1123	3.098	1.056	0.041	0.065	1.063
Cold-formed SHS and RHS	1128	3.098	1.085	0.054	0.072	1.041