Structural performance of cold-formed high strength steel circular hollow sections under combined compression and bending

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Abstract

The ultimate strengths of cold-formed high strength steel (CFHSS) circular hollow sections (CHS) beam-columns have been experimentally and numerically studied in this investigation. The steel grades of CHS members were S700, S900 and S1100 with the nominal 0.2% proof stresses of 700, 900 and 1100 MPa, respectively. In the experimental program, 32 CHS short beam-columns were axially compressed. The test results obtained in this study were used to develop an accurate finite element (FE) model. The developed FE model precisely replicated the overall structural behaviour of CHS beam-column test specimens. After validation, a detailed parametric study was conducted. The test results of CFHSS CHS long beam-columns reported by Ma et al. (2021) were also included in this study. In the parametric study, 150 long and short beam-columns (LBC and SBC) were analysed. The values of diameter-to-thickness ratio of CHS members varied from 9.7 to 108. The ultimate compression capacities of 190 test and numerical data were compared with the nominal compression capacities predicted from American, Australian and European standards. The safety levels of design rules given in these standards were examined by performing a reliability analysis. It is

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shown that the American specification provided the closest, least scattered and reliable predictions. On the contrary, the Australian standard provided very conservative, highly scattered and unreliable predictions. In addition, the predictions from European code were also quite scattered and unreliable. Hence, it is proposed to use the American specification for the design of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100.

Keywords: *Beam-columns; CHS; Cold-formed high strength steel; Finite element; Reliability analysis; Tubular members.*

Introduction

High strength steel (HSS) is gaining widespread popularity owing to various evident merits, including enhanced strength-to-weight ratio and improved toughness. Moreover, owing to stiff and slender members, the application of HSS also appreciably curtails the cost, time and man power required in fabrication, handling, transportation and on-site works. Structural members made of cold-formed steel hollow sections are rapidly used in construction projects because of their enhanced torsional strength, low drag coefficients, elegant appearance, and their ability to fill the hollow space with concrete (Xu et al. 2017; Li and Young 2018; Zhao et al. 2016a; Fang et al. 2021). Due to construction automation and significant advancements in the steel industry, cold-formed high strength steel (CFHSS) hollow section members with nominal yield stresses up to 1100 MPa are now available. However, it should be stressed that, currently, HSS material is mainly used in the automobile and aviation industries. The lack of economic and reliable design rules is one of the primary reasons hampering the structural applications of CFHSS tubular members on a large scale. Nonetheless, some research had been conducted investigating the performance of HSS at material (Ma et al. 2015; Hu et al. 2020; Pandey and Young, 2021a), member (Yu and Tall 1968; Usami and Fukumoto 1982; Gao et al. 2009; Ban et al. 2013; Liew et al. 2016; Ma et al. 2017 and 2018; Wang and Gardner 2017; Jiang et al. 2018) and joints (Pandey and Young 2021b and 2022) level.

The structural members are referred to as beam-columns when axial and bending loads act on such members at the same time. Beam-columns are common and essential members of various structures that are predominantly subjected to various load combinations at the same time. Although, design rules given in the latest editions of popular international standards (AISC 360 (2016); AS 4100-A1 (2012), EN 1993-1-12 (2007)) are applicable for steels with steel grades up to S690 and S700. However, the design rules were originally developed for members made of normal strength steel grades (yield stress less than or equal to 460 MPa). Literature review confirmed that except for the investigations carried out by Ma et al. (2019 and 2021), no other study is available on CFHSS hollow tubular beam-columns with steel grades higher than or equal to S700. Compared to normal strength steel, the stress–strain curve of CFHSS is quite different. The absence of yield plateau, gradual yielding, different extent of strain hardening, and low ultimate-to-yield strength ratio could change the response of HSS tubular members compared to their mild steel counterparts (Pandey and Young, 2021c). However, the early investigation on S690 steel built-up I- and box-section beam-columns were carried out by Yu and Tall (1968) and Usami and Fukumoto (1982). In addition, the ultimate axial resistances of concrete filled HSS tubular beam-columns were investigated by Liew et al. (2016), where yield stresses of tubular steel members ranged from 300 to 779 MPa, and compressive strengths of concrete mixes ranged from 52 to 193 MPa. It is, therefore, necessary to comprehensively investigate the ultimate strengths of CFHSS circular hollow sections (CHS) beam-columns by duly examining the applicability of current design provisions given in AISC 360 (2016), AS 4100 (1998) and EN 1993-1-1 (2005) standards. In this study, both experimental and numerical methods were employed to investigate the ultimate strengths of CFHSS CHS beam-columns. The test results generated in this study were used to develop an accurate finite element (FE) model. In order to enlarge the data pool, an extensive parametric study was performed using the

FE model validated in this study as well as CHS long beam-column FE model developed by Ma et al. (2021). The ultimate compression capacities of test and FE CHS beam-column specimens were compared with the nominal capacities predicted from AISC 360 (2016), AS 4100 (1998) and EN 1993-1-1 (2005) standards. It has been demonstrated that the current design rule given in AISC 360 (2016) provided the closest, least scattered and reliable predictions for the design of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100.

Experimental investigation

Test specimens and labelling

In the experimental investigation, CHS members of S900 and S1100 steel grades were used as the test specimens. In total, five different circular cross-sections were used in the experimental program to fabricate beam-column test specimens. Hot-rolled HSS plates of S900 and S1100 steel grades were cold-formed and welded using induction welding process to obtain CHS members. In this investigation, CHS members of S900 steel grade were denoted by ‘V’ series, while those made of S1100 steel grade were denoted by ‘S’ series. For the range of test specimens investigated in this study, the values of nominal diameter and wall thickness varied from 89 to 139 mm and 3 to 6 mm, respectively. The nominal dimensions ($D \times t$) of CHS were 89×3, 89×4, 108×4, 133×4 and 139×6, where D and t represent cross-section diameter and wall thickness, respectively. The test specimens were first extracted from the batch of tubes, and subsequently, the saw-cut cross-section ends were then milled to obtain flat ends. The test specimens with flat ends were then welded to end plates of 25 mm thickness.

In this study, 32 tests were conducted to examine the ultimate strengths of CFHSS CHS short beam-columns (SBC). In addition, Ma et al. (2021) also conducted tests to investigate the ultimate strengths of CFHSS CHS long beam-columns (LBC). The lengths of all LBC test specimens were 1480 mm, while the lengths of SBC test specimens were kept between 2*D* to 2.5*D*. The test specimens were grouped in accordance with the diameter of the CHS member. For each group, one test specimen was loaded concentrically to obtain pure compression capacity of the column, while for the remaining test specimens of a group, axial compression loads were applied with different values of initial loading eccentricities. The average values of measured length (L), diameter (D) and wall thickness (t) of test specimens are detailed in Table 1. In order to obtain a load-moment interaction curve for CFHSS CHS members, the test specimens were eccentrically compressed with various values of loading eccentricities that ranged from 6 mm to 110 mm. In the labelling of specimens, the first letter denotes the series, where specimens with nominal steel grades of S700, S900 and S1100 were denoted by H, V and S series, respectively, followed by nominal diameter (D) and wall thickness (t) of the CHS member. The term BC (i.e. beam-column) was added to the label. Finally, the nominal value of initial loading eccentricity (e) was then suffixed to the label. The repeated test was represented by adding the letter 'R' in the label.

Mechanical properties

The hollow section members used in this study belonged to the same batch of tubes used in other investigations conducted by Ma et al. (2015, 2016, 2017, 2019, 2021). Table 2 summarises the material properties of CHS members used in this study, and it includes initial Young's modulus (E), 0.2% proof stress ($\sigma_{0.2}$), ultimate stress (σ_u), and fracture strain (ε_{25mm}). In order to determine the material properties, tensile tests were conducted on coupon specimens taken from locations that were 90° to the seam weld of CHS member. It is worth noting that the static stress-strain curves were used to calculate the material properties of CHS members. Every tensile coupon test was paused for 90 seconds near the ultimate and in the post-ultimate region of the test curve. The load drops captured during the pauses were used to convert a test curve into a static curve.

Initial geometric imperfection measurements

The initial geometric imperfections are generally present in all tubular members. If present in significant magnitude, they could adversely affect the ultimate strengths of tubular members. Therefore, both global and local imperfections were carefully measured for LBC and SBC test specimens. With regard to LBC test specimens, Ma et al. (2021) assumed a three-point arc as the global imperfection shape. In order to measure the global imperfection shape as precise as possible, a total station was used to measure the readings at the mid-length as well as at both ends of LBC test specimens. As shown in Table 3, the mean value of measured global geometric imperfections (δ_0) at the mid-lengths of CHS LBC test specimens is L/7117. The initial local geometric imperfections of 3 CHS members (V89×3, S89×4 and S139×6) were measured by Ma et al. (2016). The specimens for local imperfection measurement were taken from the same batch of CHS members used for beam-column investigation. The imperfection readings were noted at every 5 mm interval along the length of CHS members using a calibrated linear variable displacement transducer (LVDT). In order to avoid the effects of cold-sawing on local imperfection, readings were not taken on the 50 mm region at each end of these CHS members. It should be noted that the same procedure was also successfully adopted in other studies (Young and Ellobody 2006; Zhao et al. 2016a; Buchanan et al. 2020).

The imperfection readings were measured at every 5 mm interval along the face 1, 2 and 3, as shown in Fig. 1. The maximum measured local geometric imperfection (δ) values of V89×3, S89×4 and S139×6 members are summarised in Table 4. The values of δ/t ratio are also detailed in Table 4. In this investigation, the initial local geometric imperfection had been correlated with δ/β ratio. Therefore, the values of δ were further normalized with the diameter-to-thickness ratio (β) of the corresponding CHS members, where $\beta=D/t$. The measured local geometric imperfection profile of S139×6 member is shown in Fig. 2. For other 3 CHS members (V89×4, S108×4 and S133×4) whose local geometric imperfections were not measured, the approximate values of their maximum local geometric imperfection were obtained by multiplying β of these CHS members with the mean value of δ/β. For test specimens with 'V' series, the mean value of δ/β ratio was 0.0058, while it was 0.0051 for 'S' series test specimens.

Beam-column tests

The CFHSS CHS beam-column specimens were tested in servo-controlled hydraulic testing machines using the displacement-control loading. The test setups of LBC and SBC specimens are shown in Fig. 3. The end plates of test specimens were connected to their corresponding wedge plates using bolted connections. In order to provide a pin-end boundary condition, a sharp edge block was pre-attached to each wedge plate. The wedge plate at each end was then coupled to its corresponding pit plate. Owing to this arrangement, the specimen freely rotated in the bending plane with pin-ended boundary conditions. Both the wedge plates were machined with slot holes, and the test specimens were adjusted along the slots to achieve different initial loading eccentricities. Consequently, LBC and SBC test specimens were then subjected to uniform bending moments along their lengths. The testing machine was fixed at the top, and a pit plate was connected at this position. On the other hand, the lower pit plate was connected to the movable ram. After proper alignment, the test specimen was preloaded up to approximately 3 kN using the load-controlled method. After preloading, displacement-controlled method was used to compress the LBC and SBC test specimens at 0.5 mm/min and 0.3 mm/min, respectively. Three calibrated LVDTs were used to measure axial shortening and rotation at the loading end. Moreover, two calibrated LVDTs were used to measure mid-length bending deflection of LBC and SBC test specimens. The testing machine was paused for 90 seconds near the ultimate and in the post-ultimate regions of the test curve. The load drops captured during the pauses were used to convert a test curve into a static curve. An advanced data logger was used to record the measured values of applied load (P), LVDT and strain gauge readings. A similar test setup had been successfully used in other investigation conducted on columns and beam-columns (Fang and Chan 2019; He et al. 2020; Li and Young 2021).

Measurements of initial loading eccentricities

Before testing, two strain gauges were attached to the most compression and tensile locations at the mid-length of the test specimen. The test specimen was then meticulously installed on the wedge plates using the nominal value of initial loading eccentricity (e). The test specimen together with the wedge plates, were then carefully coupled to the top and bottom pit plates. The test specimen was then preloaded and slightly adjusted to meet the nominal value of initial loading eccentricity (e) as precisely as possible. In this investigation, the measured values of loading eccentricities were determined using strain gauges and total station. In the strain gauge method, the curvature (κ) at the mid-length of test specimen undergoing uniform bending was determined using strain gauges. Once the curvature (κ) is known, the bending moment ($EI\kappa$) can be determined using the flexural formula. If, at the mid-length of test specimen, δ_y is the mean value of lateral deflection calculated from the values of two horizontal LVDTs and δ_0 is the initial global geometric imperfection, then the loading eccentricity can be determined using Eq. (1).

$$e+\delta_0=\frac{EI\kappa}{P}-\delta_y \tag{1}$$

On the other hand, using total station, the mid-length eccentricity was determined by taking the relative difference of centre coordinates of the test specimen and sharp edge. The same approach had been successfully used in other investigation conducted on beam-columns (Zhao et al. 2016a; Buchanan et al. 2020; Li and Young 2021). In this study, the strain gauge method was used when the nominal value of initial loading eccentricity was less than or equal to 50 mm. On the contrary, nominal eccentricities greater than 50 mm were measured using the total station method.

Test results and failure modes

Static load-deflection and load-end rotation curves were used to obtain test results in this study. As a result, the obtained test results were free from the influence of the applied strain rate. The test results, including ultimate compression loads (P_u) and corresponding end moments

($M_{end,u}$) and mid-length moments ($M_{mid,u}$), are detailed in Tables 5 and 6 for LBC and SBC test specimens, respectively. Ma et al. (2021) reported that all LBC test specimens were failed by global buckling and showed significant second-order effects under applied axial compression loads. For V89×3 series, the failed test specimens are shown in Fig. 4. As the cross-sections of LBC test specimens were compact, therefore no local buckling was noticed during the tests. Moreover, the influence of local geometric imperfections was found negligible on all LBC test specimens (Ma et al. 2021). The load-mid length deflection curves of V89×3 LBC test specimens are shown in Fig. 5. With regard to SBC test specimens, section yielding and local buckling were generally the observed failure modes. The influence of second-order effects and global geometric imperfections were negligible on all SBC test specimens. Fig. 6 presents the typical load-end rotation curves of V89×3 series of SBC test specimens. It should be noted that the pure flexural capacities of CFHSS CHS members were determined by Ma et al. (2017) using four-point bending tests that based on the same batch of test specimens in this study. Therefore, the ultimate flexural capacities (M_u) of CHS members can be referred to Ma et al. (2017).

Numerical investigation

General

This section presents the details of the numerical program conducted on CFHSS CHS beam-columns. ABAQUS (2017) was used to carry out a detailed numerical investigation. The measured material properties, member dimensions and eccentricities were used to develop accurate FE models. Additionally, measured initial geometric imperfections were also included in the developed FE models. Moreover, the influence of residual stresses has also been discussed.

Material modelling and element type

A shell element with four nodes and double curvature along with reduced integration feature (i.e., S4R) has been proved accurate and efficient to simulate various metallic materials and cross-sections subjected to different loading cases (Xu et al. 2017; Li and Young 2019; Li and Young 2022). Therefore, in this study, long and short beam-columns were modelled using S4R element. A mesh sensitivity analysis was conducted, and finally, seedings at a spacing of $D/15$ along the longitudinal and transverse directions of the specimen were found accurate enough for the precise replication of the test results. Ma et al. (2015) proposed a constitutive stress-strain model for CFHSS hollow section members, which was adopted in this investigation for material modelling. However, before assigning the material properties to FE models, the engineering stress-strain curve was converted into true stress (σ_{true}) and logarithmic plastic strain ($\varepsilon_{true,pl}$) curve using the conversion equations given in the design manual of ABAQUS (2017).

During the cold-working process, bending and membrane residual stresses are introduced in the cold-formed sections. The pattern and magnitude of these residual stresses were comprehensively investigated by Ma et al. (2015) along both longitudinal and transverse directions of CFHSS tubular members. Ma et al. (2015) reported significant presence of bending residual stresses along the longitudinal direction of CFHSS tubular members. On the contrary, membrane residual stresses were present in small amounts along both longitudinal and transverse directions of CFHSS tubular members. The bending stresses released during the cutting of coupon specimens from tubular members were restored during the tensile tests of coupon specimens (Rasmussen and Hancock 1993). Therefore, the constitutive material model developed by Ma et al. (2015) had implicitly included the effects of bending residual stresses. On the other hand, it has been concluded in many studies (Huang and Young 2015; Pandey et al., 2021a and 2021b) that membrane residual stresses have negligible effects on the overall structural behaviour of different cold-formed metallic members subjected to various loading conditions. The maximum values of measured longitudinal membrane residual stresses reported by Ma et al. (2015) were less than 20% of the yield stresses of investigated CFHSS tubular members, and their effects were negligible on the accuracies of FE models (Ma et al. 2017 and 2018). Therefore, in order to keep the FE modelling methodology simple, the residual stresses were not separately included in the numerical models of CFHSS CHS beam-columns.

Load and boundary conditions

Both LBC and SBC test specimens were subjected to axial compression loads. As shown in Fig. 7, the pin-end boundary condition of CHS beam-column was achieved using a parallel sharp pointed edge between wedge and pit plates. Therefore, for CHS beam-columns with zero nominal eccentricity (i.e. e=0), a reference point was kinematically coupled to the surface of each end cross-section. The effective length (L_e) of the test specimen is shown in Fig. 7 and is equal to the length of specimen (L) and the total heights of wedge and end plates at both ends. For both LBC and SBC test specimens, the total height of pin, wedge plate and end plate was 87.5 mm, therefore, reference points were positioned at 87.5 mm from the cross-section end. For beam-column specimens with eccentricity, the two reference points were then laterally offset in the bending plane by a distance equal to the measured loading eccentricity. In order to exactly reciprocate experimental boundary conditions, both top and bottom reference points were allowed to rotate in the bending plane. In addition, the bottom reference point was also allowed to translate along the length of the specimen. All other degrees of freedom (DOF) of the top and bottom reference points were restrained. Moreover, the DOF of all other nodes of LBC and SBC specimens were kept unrestrained. Using displacement control method, axial compression load was then applied at the bottom reference point of the specimen by duly following the static RIKS procedure given in ABAQUS (2017). The size of step increment was kept small in the RIKS procedure in order to obtain smooth load-deflection and load-end rotation curves. In addition, the parameter that enables the FE model to undergo large non-linear deformation (*NLGEOM) was activated during the FE analysis.

Inclusion of initial geometric imperfections

The global and local geometric imperfections were incorporated in the FE models of LBC and SBC specimens, respectively. The global and local geometric imperfection profiles were obtained by conducting elastic buckling analyses on identical FE specimens. The BUCKLE command of ABAQUS (2017) was used to implement this methodology. Through elastic buckling analysis, which is also termed as eigenvalue analysis, the global and local buckling imperfection profiles were separately obtained for LBC and SBC specimens, respectively. The first mode of elastic buckling analysis of FE specimen was treated as the imperfection mode of that specimen. The deformation scale of the first buckling mode was then ramped up to the measured values of geometric imperfections. The scaled eigenmode shape was then superimposed on the FE model. The effect of local geometric imperfection was negligible on the ultimate compression capacities of LBC numerical specimens (Ma et al. 2021). On the other hand, in this study, the effect of global geometric imperfection was found negligible on the ultimate compression capacities of SBC numerical specimens. The approach adopted in this study for the inclusion of initial geometric imperfection was consistent with the approach adopted in other similar investigation on beam-columns (Huang and Young 2015; Zhao et al. 2016b; Li and Young 2022).

Validation of finite element models

The modelling methodologies detailed earlier were used to develop and validate FE models of long and short beam-columns. The LBC finite element model was validated by Ma et al. (2021). On the other hand, the validation of SBC finite element model was carried out using the test results obtained in this study. For validation, the ultimate compression capacities of LBC and SBC test specimens ($P_{Exp,LBC}$, $P_{Exp,SBC}$) were compared with those predicted from their corresponding FE models ($P_{FE,LBC}$, $P_{FE,SBC}$). During the validation process, failure modes, load-deflection and load-end rotation curves were also compared between all test and FE specimens. Table 7 presents the comparisons of ultimate compression capacities of LBC test specimens ($P_{Exp,LBC}$) with those predicted from corresponding FE models ($P_{FE,LBC}$). The values of mean and coefficients of variation (COV) of this comparison are 0.93 and 0.034, respectively. Moreover, for V89×3-BC-e150 long beam-column, the failure mode comparison between the test and FE model is shown in Fig. 8. From Table 7 and Fig. 8, it is shown that the FE model was able to predict the overall structural behaviour of LBC test specimens. With regard to SBC specimens, Table 8 presents the comparisons of ultimate compression capacities between tests ($P_{Exp,SBC}$) and FE ($P_{FE,SBC}$) models. For this comparison, the mean value of test-to-FE ratios of ultimate compression capacities is 0.97, and the corresponding COV is 0.048, as shown in Table 8. On the other hand, for short beam-columns, the load-end rotation and failure mode comparisons between test and FE model are shown in Figs. 9 and 10, respectively. From these comparisons, it is apparent that the validated SBC FE model precisely replicated the overall structural behaviour of CFHSS CHS short beam-columns. The existing errors between test and FE predictions could be due to the uneven distribution of geometric imperfections in the test specimens.

Parametric study

A detailed parametric study was conducted on CFHSS CHS beam-columns with an aim to enlarge the data pool by duly including a large range of D/t ratio. In total, 150 specimens, including 75 LBC and 75 SBC specimens, were analysed in the parametric study. The FE models with nominal yield stresses of 700 MPa were denoted by H series, whereas the explanations for V and S series have been mentioned earlier in the paper. Each series of both LBC and SBC specimens included 15 CHS sections, wherein for each section, axial compression was applied with one concentric and four eccentric loadings. The FE methodologies used in the validation of CHS long and short beam-column models were also adopted in their corresponding parametric studies. With regard to material modelling, the constitutive stress-strain model developed by Ma et al. (2015) was also used in the parametric study. The cross-section diameter of CHS beam-column specimens varied from 48.3 mm to

323.9 mm, while the values of D/t varied from 9.7 to 108. In the parametric study, the lengths of all LBC and SBC specimens were kept equal to 2800 mm and 600 mm, respectively. In the LBC FE model, a value of $L/1500$ was taken as the global geometric imperfection. On the other hand, in the SBC FE model, the value of local geometric imperfection was considered in accordance with the methodology explained earlier in the paper. The effective lengths of all FE specimens were equal to the sum of the lengths of specimens (L) and total heights of end and wedge plates at each end of the specimens (87.5 mm), i.e., $L_e = L + 2\times87.5$ mm.

In the parametric study, short beam-columns with large member slenderness were generally failed by global buckling, while remaining short beam-columns were generally failed by either local buckling or section yielding. On the other hand, global buckling was the dominant failure mode for long beam-columns with compact cross-sections, while the long beam-columns with large values of D/t were generally failed by interaction of global and local buckling. The ultimate compression capacities (P_u) of CHS long beam-column specimens obtained from tests (Ma et al. 2021) and numerical investigation conducted in this study are detailed in Table 9. On the other hand, Table 10 presents the ultimate compression capacities (P_u) of CHS short beam-column specimens obtained from tests and parametric study carried out in this investigation. The FE specimens of the parametric study were labelled as "series, *D*, *t, LBC* or *SBC*, *e*". In the labelling, series represent for H, V and S series; followed by cross-section diameter (D), and wall thickness (t) of CHS members; *LBC* and *SBC* stand for long and short beam-columns, respectively; and e stands for the nominal value of loading eccentricity. The value of initial loading eccentricity of CHS beam-column specimens ranged from $D/8$ to D

in the parametric study.

Current design provisions

General

Although the design provisions given in the latest editions of AISC 360 (2016), AS 4100-A1 (2012) and EN 1993-1-12 (2007) permit the use of steels with nominal steel grades up to S690 and S700, however, it is worth noting that the design recommendations given in the specifications (AISC 360 (2016), AS 4100 (1998) and EN 1993-1-1 (2005)) are mainly developed from the research conducted on specimens made of normal strength steel. Therefore, it becomes essential to examine the appropriateness of current design rules given in these specifications for CFHSS CHS beam-columns.

AISC 360 (2016)

In order to design a beam-column member, a two-phase interactive relationship is given in AISC 360 (2016), as shown in Eq. (2). In this interaction formula, the ratio of ultimate compression load (P_u) normalised with nominal predicted pure compression capacity (P_n) is linearly proportioned to the ratio of second-order bending moment (M_u) normalised with nominal predicted pure bending capacity (M_n). For members subject to uniaxial bending and possess single uniform curvature along their length, M_u can be determined by amplifying the end moment ($M_{end,u}$), as shown in Eq. (3). The term P is the applied load, and P_{cr} is the Euler's critical buckling load in Eq. (3).

$$\begin{cases} \frac{P_\mathrm{u}}{P_\mathrm{n}} + \frac{8}{9}\frac{M_\mathrm{u}}{M_\mathrm{n}} \le 1 & \text{for} \quad \frac{P_\mathrm{u}}{P_\mathrm{n}} \ge 0.2 \\ \\ \frac{P_\mathrm{u}}{2P_\mathrm{n}} + \frac{M_\mathrm{u}}{M_\mathrm{n}} \le 1 & \text{for} \quad \frac{P_\mathrm{u}}{P_\mathrm{n}} < 0.2 \end{cases} \tag{2}$$

$$M_\mathrm{u} = \frac{M_\mathrm{end,u}}{(1 - P/P_\mathrm{cr})} \tag{3}$$

AS 4100 (1998)

The interactive relationship given in AS 4100 (1998) standard for the design of a beam-column member is shown in Eq. (4). A more simplified version of Eq. (4) is shown in Eq. (5). Similar to the design rule given in AISC 360 (2016), the ratio of ultimate compression load (P_u) normalised with nominal predicted pure compression capacity (P_n) is linearly proportioned to the ratio of second-order bending moment (M_u) normalised with nominal predicted pure bending capacity (M_n). Similar to AISC 360 (2016), M_u can be calculated using the amplification factor shown in Eq. (3). Moreover, as the cold-formed sections used in this study were non-stress relieved, therefore, a compression member section constant (α_b) equal to -0.50 was adopted to calculate the nominal pure compression capacity (P_n) of the beam-column specimen.

$$M_\mathrm{u} \le M_\mathrm{n}\left(1 - \frac{P_\mathrm{u}}{P_\mathrm{n}}\right) \tag{4}$$

$$\frac{P_\mathrm{u}}{P_\mathrm{n}} + \frac{M_\mathrm{u}}{M_\mathrm{n}} \le 1 \tag{5}$$

EN 1993-1-1 (2005)

The beam-column design method given in EN 1993-1-1 (2005) is similar to those adopted in AISC 360 (2016) and AS 4100 (1998) specifications. However, unlike the linear interaction formulae given in AISC 360 (2016) and AS 4100 (1998), the design rule given in EN 1993-1-1 (2005) uses an interaction factor (k_{yy}) to consider the non-linear beam-column interaction relationship, as shown in Eq. (6). Unlike the use of second-order design moment in the beam-column design rules given in AISC 360 (2016) and AS 4100 (1998) specifications, the first-order design moment is used in the interaction formula given by Eq. (6) (Ziemian 2010). Therefore, $M_{end,u}$ has been used in Eq. (6) instead of M_u. The calculation of interaction factor (k_{yy}) is demonstrated through two methods in EN 1993-1-1 (2005) code. The method-1 given in the Annex-A of EN 1993-1-1 (2005) is more accurate, and therefore, it was used in this study to calculate k_{yy}. Furthermore, buckling curve 'c' given in EN 1993-1-1 (2005) and recommended for cold-formed hollow sections was used to calculate the nominal capacities under pure compression (P_n).

$$\frac{P_{\mathrm{u}}}{P_{\mathrm{n}}} + k_{\mathrm{yy}} \frac{M_{\mathrm{end,u}}}{M_{\mathrm{n}}} \leq 1 \tag{6}$$

Reliability Analysis

In this investigation, the reliability of current design rules is checked by performing a reliability analysis detailed in AISI S100 (2016). The target reliability index (β_0) was determined as per Section K2.1.1 of AISI S100 (2016). In this investigation, a lower bound limit of 2.50 was used as the target reliability index (β_0). Therefore, when the value of β_0 was greater than or equal to 2.50, the design rule was treated as reliable in this study. The reliability index (β_0) was calculated as follows:

$$\beta_0 = \frac{\ln(C_\phi M_m F_m P_m / \phi)}{\sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \tag{7}$$

In Eq. (7), the calibration coefficient (C_ϕ) was calculated using a dead load (DL)-to-live load (LL) ratio of 0.20. For the material factor, the mean value and COV were denoted by M_m and V_M, respectively. On the other hand, for the fabrication factor, the mean value and COV were denoted by F_m and V_F, respectively. The resistance factor required to convert nominal strength to design strength was denoted by ϕ. The mean value of ratios of test and FE strengths of specimens-to-nominal strengths predicted from code was denoted by P_m, while the corresponding COV was denoted by V_P. A correction factor (C_P) recommended in AISI S100 (2016) was also used to consider the influence of data size. Besides, the COV of the load effects was represented by V_Q.

Referring to the Section K2.1.1 of AISI S100 (2016), the values of M_m and V_M were taken as 1.1 and 0.10, respectively. Additionally, in the calculation of β_0, the values of F_m and V_F were taken as 1.0 and 0.05, respectively. Besides, the values of P_m and V_P can be obtained from Tables 9-11. For the purpose of evaluating the appropriateness of the design rule given in AISC 360 (2016), the load combination was taken as 1.2DL + 1.6LL, while the values of the calibration coefficient (C_ϕ) and resistance factor (ϕ) were taken as 1.521 and 0.90, respectively. Further, to assess the suitability of the design rule given in AS 4100 (1998), the load combination was taken as 1.2DL + 1.5LL, while the values of calibration coefficient (C_ϕ) and resistance factor (ϕ) were taken as 1.445 and 0.90, respectively. In order to examine the adequacy of the design rule given in EN 1993-1-1 (2005) for CFHSS CHS beam-columns, the load combination was taken as 1.35DL + 1.5LL, while the values of calibration coefficient (C_{ϕ}) and resistance factor (ϕ) were taken as 1.463 and 1.0, respectively.

Evaluation of Current Design Provisions

The comparisons of ultimate compression capacities obtained from the experimental and numerical investigation with nominal capacities predicted from AISC 360 (2016), AS 4100 (1998) and EN 1993-1-1 (2005) are discussed in this section of the paper. A total of 190 data was used for the comparison, including 8 LBC test strengths (Ma et al. 2021), 32 SBC test strengths, 75 LBC numerical strengths and 75 SBC numerical strengths. In order to predict the strengths from the interaction curves of AISC 360 (2016), AS 4100 (1998) and EN 1993-1-1 (2005) specifications, the data points on the load-moment interaction curves of these specifications were determined using a line intersecting these curves with slope equal to the initial loading eccentricity (e) of the specimen, as shown in Fig. 11. The terms P_y and M_p respectively represent yield load and plastic moment of the cross-section of tubular member in Fig. 11. The comparisons of test and FE ultimate compression capacities with nominal predicted capacities for LBC and SBC specimens are shown in Tables 9 and 10, respectively. The comparisons are also graphically shown in Figs. 12-14.

The comparisons of experimental and numerical ultimate compression capacities (P_u) of CFHSS CHS long and short beam-columns with nominal capacities predicted from AISC 360 (2016) (P_{AISC}) are presented in Tables 9-11 and Fig. 12. For LBC specimens, the values of mean and COV of P_u/P_{AISC} are 1.07 and 0.069, respectively. On the other hand, for SBC specimens, the values of mean and COV of P_u/P_{AISC} are 1.08 and 0.099, respectively. For the overall comparison, including LBC and SBC specimens, the values of mean and COV of P_u/P_{AISC} are 1.07 and 0.087, respectively. In AISC 360 (2016), the values of resistance factors recommended for both axial compression (ϕ_c) and bending (ϕ_b) are equal to 0.90. Thus, the calculated value of β_0 is 2.72. Therefore, it is evident from the comparison results that the ultimate compression capacities of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100 are overall underestimated up to 7% by the design rule given in AISC 360 (2016). Furthermore, the current design rule given in AISC 360 (2016) is found reliable for the codified value of the resistance factor. Moreover, Fig. 12 demonstrated that the current design rule given in AISC 360 (2016) possesses reasonable scatter when compared with the ultimate compression capacities of CFHSS CHS beam-columns.

The comparisons of experimental and numerical ultimate compression capacities (P_u) of CFHSS CHS beam-columns with the nominal capacities predicted from AS 4100 (1998) (P_{AS}) are presented in Tables 9-11 and Fig. 13. For LBC specimens, the values of mean and COV of P_u/P_{AS} are 1.40 and 0.362, respectively. On the other hand, for SBC specimens, the values of mean and COV of P_u/P_{AS} are 1.46 and 0.352, respectively. For the overall comparison, including LBC and SBC specimens, the values of mean and COV of P_u/P_{AS} are 1.43 and 0.356, respectively. The values of resistance factors for both axial compression (ϕ_c) and bending (ϕ_b) are equal to 0.90 in AS 4100 (1998). Thus, the calculated value of β_0 is 2.14. From the overall comparison, it is evident that AS 4100 (1998) standard provides very conservative and highly scattered predictions for the ultimate compression capacities of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100.

In this study, EN 1993-1-1 (2005) was used to examine the ultimate compression capacities of CFHSS CHS beam-columns. Tables 9-11 and Fig. 14 present the comparisons of experimental and numerical ultimate compression capacities (P_u) of CFHSS CHS beam-columns with the nominal capacities predicted from EN 1993-1-1 (2005) (P_{EC3}). For LBC specimens, the values of mean and COV of P_u/P_{EC3} are 1.04 and 0.121, respectively. On the other hand, for SBC specimens, the values of mean and COV of P_u/P_{EC3} are 1.08 and 0.188, respectively. For the overall comparison, including LBC and SBC specimens, the values of mean and COV of P_u/P_{EC3} are 1.07 and 0.165, respectively. The recommended value of resistance factor for beam-column members is 1.0 in EN 1993-1-1 (2005). Thus, the calculated value of β_0 is 1.86. It is evident from the overall comparison that the design method given in EN 1993-1-1 (2005) underestimated the ultimate compression capacities of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100 by 7%. In addition, the predictions from the current design method given in EN 1993-1-1 (2005) are quite scattered and not reliable for ϕ=1.0.

Ma et al. (2017) reported that the current design rules given in AS 4100 (1998) and EN 1993-1-1 (2005) codes provide very conservative predictions for the design of CFHSS tubular beams. It should be noted that the load-moment interaction curves in different codes of practice are generally based on two reference points (i.e. pure flexural strength and pure compression strength). The significant conservative predictions of flexural strengths of CFHSS CHS beams by design rules given in AS 4100 (1998) and EN 1993-1-1 (2005) codes lead to the overall conservative and scattered predictions of CFHSS CHS beam-columns in this study. The overall summary of the comparison results is presented in Table 11. It is evident from Table 11 that AISC 360 (2016) provided the closest, least scattered and reliable predictions amongst other specifications. Although the design method given in EN 1993-1-1 (2005) also well-predicted the ultimate compression capacities of CFHSS CHS beam-columns, however, the predictions are quite scattered, and the design rule is not reliable. The design rules given in AS 4100 (1998) and EN 1993-1-1 (2005) codes remained unreliable even for ϕ=0.85. It is, therefore, suggested to use the current beam-column design rule given in AISC 360 (2016) together with the codified value of resistance factor for the design of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100.

Conclusions

The ultimate strengths of cold-formed high strength steel (CFHSS) circular hollow sections (CHS) beam-columns was experimentally and numerically investigated in this study. A total of 32 CHS short beam-columns (SBC) was tested in the experimental program. In addition, the results of 8 CHS long beam-column (LBC) tests carried out by Ma et al. (2021) were also included. In the numerical program, a finite element (FE) model was developed and validated against the test results of SBC specimens, while for LBC specimens, the FE model developed and validated by Ma et al. (2021) was used in this study. The validated FE models of CHS long and short beam-columns precisely replicated the ultimate compression capacities, load-deflection and load-end rotation curves and failure modes of CHS beam-column test specimens. Upon validation, a detailed numerical parametric study was performed, which comprised of 150 finite element models, including 75 LBC models and 75 SBC models. The steel grades of CHS members were S700, S900 and S1100. The values of *D*/*t* of the CHS members varied from 9.7 to 108.

Short beam-columns with large member slenderness were generally failed by global buckling, while remaining short beam-columns were generally failed by either local buckling or section yielding. On the other hand, global buckling was the dominant failure mode for long beam-columns with compact cross-sections, while the long beam-columns with large values of *D*/*t* were generally failed by interaction of global and local buckling. The nominal capacities predicted from AISC 360 (2016), AS 4100 (1998) and EN 1993-1-1 (2005) specifications were compared with the ultimate compression capacities obtained from the experimental and numerical analyses. It is shown that the design rule given in AISC 360 (2016) provided the closest, least scattered and reliable predictions for the ultimate compression capacities of CFHSS CHS beam-columns. On the other hand, the predictions of AS 4100 (1998) standard were very conservative, highly scattered and unreliable for CFHSS CHS beam-columns. Moreover, the predictions of EN 1993-1-1 (2005) standard were also quite scattered and unreliable. Hence, it is suggested to use the AISC 360 (2016) standard for the design of cold-formed steel CHS beam-columns with steel grades ranged from S700 to S1100.

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Data Availability Statement

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Table 1. Measured average dimensions of CHS members

Beam-columns	Sections (VD×t) (SD×t)	L (mm)	D (mm)	t (mm)
Short beam-column	V89×3	220	88.9	2.97
	V89×4	220	89.1	3.90
	S89×4	220	89.1	3.88
	S108×4	220	108.3	3.90
	S133×4	310	133.4	3.89
	S139×6	310	138.9	5.89
Long beam-column	V89×3	1480	89.0	2.94

Table 2. Measured material properties of CHS members (Ma et al. 2015)

Sections (VD×t) (SD×t)	E (GPa)	$\sigma_{0.2}$ (MPa)	σ_{u} (MPa)	ε_{25mm} (%)
V89×3	209	1053	1124	10
V89×4	210	1054	1108	12
S89×4	205	1180	1317	11
S108×4	215	1180	1292	10
S133×4	204	1159	1291	10
S139×6	194	1014	1382	10

Table 3. Measured global geometric imperfections at mid-length of CHS long beam-columns (Ma et al. 2021)

Specimens (VD×t-BC-e#)	δ_0 (mm)	δ_0/L
V89×3-BC-e0	-0.152	-1/9711
V89×3-BC-e3	0.000	0
V89×3-BC-e10	-0.114	-1/12948
V89×3-BC-e20	-0.292	-1/5067
V89×3-BC-e40	0.000	0
V89×3-BC-e40-R	-0.063	-1/23307
V89×3-BC-e80	0.152	1/9711
V89×3-BC-e150	0.889	1/1665
Mean		1/7117

Table 4. Measured local geometric imperfections of CHS members (Ma et al. 2016)

Specimens (VD×t) (SD×t)	Diameter-to-thickness ratio (β) $\beta = D/t$	Maximum imperfection δ (mm)	δ/t (%)	δ/β (mm)	$(\delta/\beta)_{avg}$ (mm)
V89×3	30.1	0.174	5.9	0.0058	0.0058
S89×4	22.7	0.072	1.9	0.0031	0.0051
S139×6	23.6	0.168	2.8	0.0071	0.0051

Table 5. Test results of long beam-columns

Specimens (VD×t-BC-e#)	$e+\delta_0$ (mm)	*P_u (kN)	*$M_{end,u}$ (kNm)	*$M_{mid,u}$ (kNm)
V89×3-BC-e0	0.28	444.4	0.2	2.2
V89×3-BC-e3	2.98	367.9	1.1	8.8
V89×3-BC-e10	10.39	300.3	3.2	13.0
V89×3-BC-e20	21.68	249.3	5.5	15.7
V89×3-BC-e40	39.72	200.9	8.0	18.1
V89×3-BC-e40-R	39.61	200.1	7.9	18.2
V89×3-BC-e80	79.85	144.3	11.5	20.7
V89×3-BC-e150	151.54	98.4	14.8	22.4

Note: * data obtained from Ma et al. (2021).

Table 6. Test results of short beam-columns

Specimens (VD×t-BC-e#) (SD×t-BC-e#)	e (mm)	P_u (kN)	$M_{end,u}$ (kNm)	$M_{mid,u}$ (kNm)
V89×3-BC-e0	0.36	772.8	0.3	0.7
V89×3-BC-e6	6.10	671.7	4.1	5.2
V89×3-BC-e20	20.14	482.8	9.7	10.8
V89×3-BC-e40	39.74	355.0	14.1	15.2
V89×3-BC-e40-R	39.83	347.4	13.8	14.9
V89×3-BC-e80	80.22	193.2	15.5	16.1
V89×4-BC-e0	0.10	1049.7	0.1	0.4
V89×4-BC-e0-R	0.23	1080.1	0.3	0.7
V89×4-BC-e6	6.12	846.5	5.2	6.4
V89×4-BC-e20	20.06	652.9	13.1	14.8
V89×4-BC-e40	39.86	468.0	18.7	20.5
V89×4-BC-e80	77.85	282.9	22.0	23.3
S89×4-BC-e0	0.42	1122.5	0.5	1.4

S89×4-BC-e6	5.82	981.0	5.7	7.7
S89×4-BC-e20	19.50	753.5	14.7	17.2
S89×4-BC-e40	38.66	539.8	20.9	23.8
S89×4-BC-e80	80.15	316.5	25.4	27.1
S108×4-BC-e0	0.10	1408.9	0.1	0.4
S108×4-BC-e10	10.31	1196.2	12.3	14.6
S108×4-BC-e25	24.71	947.8	23.4	26.0
S108×4-BC-e50	50.21	649.2	32.6	34.8
S108×4-BC-e90	89.98	394.0	35.5	36.9
S133×4-BC-e0	0.03	1858.2	0.1	1.7
S133×4-BC-e10	9.89	1508.4	14.9	17.3
S133×4-BC-e25	24.22	1240.7	30.1	33.8
S133×4-BC-e25-R	24.09	1192.7	28.7	31.2
S133×4-BC-e55	54.14	861.2	46.6	49.8
S133×4-BC-e110	108.23	508.9	55.1	58.0
S139×6-BC-e0	0.08	3107.1	0.2	6.7
S139×6-BC-e15	12.96	2521.1	32.7	44.6
S139×6-BC-e30	30.26	2048.9	62.0	75.3
S139×6-BC-e60	60.03	1357.9	81.5	91.5

Table 7. Comparisons of Test and FE ultimate compression capacities for CHS Long Beam-Columns (Ma et al. 2021)

Specimens (VD×t-BC-e#)	$P_{Exp,LBC}$ (kN)	$P_{FE,LBC}$ (kN)	$P_{Exp,LBC}/P_{FE,LBC}$
V89×3-BC-e0	444.4	488.4	0.91
V89×3-BC-e3	367.9	418.1	0.88
V89×3-BC-e10	300.3	330.0	0.91
V89×3-BC-e20	249.3	271.0	0.92
V89×3-BC-e40	200.9	216.0	0.93
V89×3-BC-e40-R	200.1	212.9	0.94
V89×3-BC-e80	144.3	150.3	0.96
V89×3-BC-e150	98.4	100.4	0.98
Mean			0.93
COV			0.034

Table 8. Comparisons of Test and FE ultimate compression capacities for CHS Short Beam-Columns.

Specimens ($VD\times t$-BC-e#) ($SD\times t$-BC-e#)	$P_{Exp,SBC}$ (kN)	$P_{FE,SBC}$ (kN)	$P_{Exp,SBC}/P_{FE,SBC}$
V89×3-BC-e0	772.8	805.4	0.96
V89×3-BC-e6	671.7	674.3	1.00
V89×3-BC-e20	482.8	487.8	0.99
V89×3-BC-e40	355.0	348.3	1.02
V89×3-BC-e40-R	347.4	346.3	1.00
V89×3-BC-e80	193.2	217.6	0.89
V89×4-BC-e0	1049.7	1111.1	0.94
V89×4-BC-e0-R	1080.1	1115.3	0.97
V89×4-BC-e6	846.5	911.5	0.93
V89×4-BC-e20	652.9	668.5	0.98
V89×4-BC-e40	468.0	479.6	0.98
V89×4-BC-e80	282.9	309.7	0.91
S89×4-BC-e0	1122.5	1263.0	0.89
S89×4-BC-e6	981.0	1058.6	0.93
S89×4-BC-e20	753.5	778.4	0.97
S89×4-BC-e40	539.8	559.7	0.96
S89×4-BC-e80	316.5	345.1	0.92
S108×4-BC-e0	1408.9	1546.3	0.91
S108×4-BC-e10	1196.2	1211.1	0.99
S108×4-BC-e25	947.8	929.4	1.02
S108×4-BC-e50	649.2	656.3	0.99
S108×4-BC-e90	394.0	444.5	0.89
S133×4-BC-e0	1858.2	1849.4	1.00
S133×4-BC-e10	1508.4	1486.5	1.01
S133×4-BC-e25	1240.7	1175.7	1.06
S133×4-BC-e25-R	1192.7	1185.5	1.01
S133×4-BC-e55	861.2	810.1	1.06
S133×4-BC-e110	508.9	514.0	0.99
S139×6-BC-e0	3107.1	3319.5	0.94
S139×6-BC-e15	2521.1	2558.5	0.99
S139×6-BC-e30	2048.9	2001.7	1.02
S139×6-BC-e60	1357.9	1438.4	0.94
Mean			0.97
COV			0.048

Table 9 Comparisons of test and FE ultimate compression capacities with nominal predicted capacities for CHS Long Beam-Columns.

Specimens (HD×t-BC or LBC-e#) (VD×t-BC or LBC-e#) (SD×t-BC or LBC-e#)	P_u (kN)	P_u/P_{AISC}	P_u/P_{AS}	P_u/P_{EC3}
V89x3-BC-e0	444.4#	1.04	1.11	1.27
V89x3-BC-e3	367.9#	1.00	1.08	1.14
V89x3-BC-e10	300.3#	1.01	1.12	1.11
V89x3-BC-e20	249.3#	1.02	1.15	1.10
V89x3-BC-e40	200.9#	1.02	1.17	1.07
V89x3-BC-e40-R	200.1#	1.02	1.18	1.08
V89x3-BC-e80	144.3#	1.03	1.22	1.06
V89x3-BC-e150	98.4#	1.04	1.27	1.05
H48.3x5-LBC-e0	37.5	1.11	1.08	1.11
H48.3x5-LBC-e2	36.6	1.13	1.11	1.09
H48.3x5-LBC-e5	34.7	1.13	1.11	1.06
H48.3x5-LBC-e16	30.5	1.11	1.11	1.01
H48.3x5-LBC-e48	24.2	1.06	1.08	0.98
H88.9x5-LBC-e0	268.2	1.10	1.12	1.25
H88.9x5-LBC-e3	239.0	1.09	1.11	1.16
H88.9x5-LBC-e9	211.5	1.08	1.11	1.11
H88.9x5-LBC-e30	162.6	1.03	1.07	1.06
H88.9x5-LBC-e90	107.1	0.96	1.02	1.02
H168.3x4-LBC-e0	1066.8	1.05	1.14	1.22
H168.3x4-LBC-e6	879.2	1.00	1.09	1.11
H168.3x4-LBC-e17	737.4	1.00	1.12	1.11
H168.3x4-LBC-e56	510.0	1.01	1.17	1.14
H168.3x4-LBC-e168	286.3	0.99	1.22	1.17
H219.1x3-LBC-e0	1060.2	1.00	1.26	0.98
H219.1x3-LBC-e7	922.2	0.97	1.25	0.96
H219.1x3-LBC-e22	796.0	0.99	1.31	1.00
H219.1x3-LBC-e73	542.6	1.00	1.40	1.06
H219.1x3-LBC-e219	289.4	0.97	1.46	1.09
H323.9x3-LBC-e0	1487.5	0.90	1.35	0.76
H323.9x3-LBC-e11	1356.0	0.92	1.40	0.79
H323.9x3-LBC-e32	1180.8	0.97	1.50	0.84
H323.9x3-LBC-e108	775.9	0.99	1.62	0.90
H323.9x3-LBC-e324	389.0	0.98	1.70	0.91
V48.3x5-LBC-e0	36.6	1.11	1.05	1.08
V48.3x5-LBC-e2	36.0	1.12	1.08	1.07
V48.3x5-LBC-e5	34.5	1.12	1.09	1.05

V48.3x5-LBC-e16	31.1	1.11	1.09	1.01
V48.3x5-LBC-e48	25.9	1.09	1.08	0.98
V88.9x5-LBC-e0	263.7	1.11	1.08	1.20
V88.9x5-LBC-e3	242.6	1.11	1.11	1.14
V88.9x5-LBC-e9	220.0	1.11	1.12	1.09
V88.9x5-LBC-e30	176.5	1.08	1.10	1.04
V88.9x5-LBC-e90	124.7	1.04	1.08	1.04
V168.3x4-LBC-e0	1247.5	1.09	1.21	1.27
V168.3x4-LBC-e6	1037.4	1.04	1.18	1.15
V168.3x4-LBC-e17	877.3	1.05	1.22	1.13
V168.3x4-LBC-e56	618.2	1.04	1.29	1.13
V168.3x4-LBC-e168	360.1	1.03	1.36	1.16
V219.1x3-LBC-e0	1351.5	1.05	1.41	1.04
V219.1x3-LBC-e7	1145.1	1.01	1.37	0.98
V219.1x3-LBC-e22	980.9	1.04	1.43	1.01
V219.1x3-LBC-e73	663.1	1.06	1.52	1.03
V219.1x3-LBC-e219	357.5	1.05	1.60	1.04
V323.9x3-LBC-e0	1791.5	1.02	2.03	0.72
V323.9x3-LBC-e11	1707.2	1.10	2.22	0.77
V323.9x3-LBC-e32	1472.3	1.16	2.39	0.82
V323.9x3-LBC-e108	952.4	1.19	2.60	0.84
V323.9x3-LBC-e324	473.5	1.19	2.75	0.83
S48.3x5-LBC-e0	36.2	1.11	1.03	1.08
S48.3x5-LBC-e2	35.5	1.12	1.06	1.07
S48.3x5-LBC-e5	33.9	1.11	1.07	1.04
S48.3x5-LBC-e16	30.5	1.09	1.06	0.99
S48.3x5-LBC-e48	25.6	1.07	1.06	0.96
S88.9x5-LBC-e0	260.1	1.11	1.07	1.19
S88.9x5-LBC-e3	237.7	1.10	1.08	1.11
S88.9x5-LBC-e9	216.2	1.10	1.09	1.07
S88.9x5-LBC-e30	175.8	1.07	1.09	1.02
S88.9x5-LBC-e90	128.2	1.05	1.09	1.03
S168.3x4-LBC-e0	1245.9	1.06	1.18	1.24
S168.3x4-LBC-e6	1056.1	1.03	1.18	1.14
S168.3x4-LBC-e17	906.6	1.05	1.24	1.12
S168.3x4-LBC-e56	649.9	1.06	1.32	1.13
S168.3x4-LBC-e168	388.3	1.05	1.42	1.17
S219.1x3-LBC-e0	1456.0	1.07	1.47	1.07
S219.1x3-LBC-e7	1213.4	1.03	1.42	0.99
S219.1x3-LBC-e22	1041.8	1.08	1.53	1.01
S219.1x3-LBC-e73	708.9	1.11	1.68	1.03
S219.1x3-LBC-e219	381.1	1.12	1.81	1.03
S323.9x3-LBC-e0	2149.7	1.24	2.67	0.81

S323.9x3-LBC-e11	1840.9	1.20	2.62	0.78
S323.9x3-LBC-e32	1582.4	1.26	2.81	0.82
S323.9x3-LBC-e108	1023.1	1.30	3.05	0.84
S323.9x3-LBC-e324	504.2	1.29	3.20	0.82
Mean (P_m)		1.07	1.40	1.04
COV (V_P)		0.069	0.362	0.121
Resistance Factor (ϕ)		0.90	0.90	1.00
Reliability Index (β_0)		2.77	2.05	1.91

Note: # denotes data from Ma et al. (2021).

Table 10. Comparisons of test and FE ultimate compression capacities with nominal predicted capacities for CHS Short Beam-Columns.

Specimens (HD×t-BC or SBC-e#) (VD×t-BC or SBC-e#) (SD×t-BC or SBC-e#)	P_u (kN)	P_u/P_{AISC}	P_u/P_{AS}	P_u/P_{EC3}
V89x3-BC-e0	772.8	1.03	1.17	0.98
V89x3-BC-e6	671.7	1.09	1.26	1.08
V89x3-BC-e20	482.8	1.09	1.31	1.18
V89x3-BC-e40	355.0	1.12	1.38	1.27
V89x3-BC-e40-R	347.4	1.10	1.36	1.25
V89x3-BC-e80	193.2	0.96	1.21	1.14
V89x4-BC-e0	1049.7	0.99	1.06	1.01
V89x4-BC-e0	1080.1	1.02	1.09	1.04
V89x4-BC-e6	846.5	0.99	1.08	1.06
V89x4-BC-e20	652.9	1.08	1.21	1.24
V89x4-BC-e40	468.0	1.09	1.25	1.31
V89x4-BC-e80	282.9	1.02	1.19	1.27
S89x4-BC-e0	1122.5	1.01	1.09	1.00
S89x4-BC-e6	981.0	1.07	1.19	1.11
S89x4-BC-e20	753.5	1.15	1.31	1.28
S89x4-BC-e40	539.8	1.15	1.35	1.34
S89x4-BC-e80	316.5	1.09	1.30	1.32
S108x4-BC-e0	1408.9	1.04	1.20	0.96
S108x4-BC-e10	1196.2	1.14	1.37	1.15

S108x4-BC-e25	947.8	1.20	1.47	1.28
S108x4-BC-e50	649.2	1.16	1.46	1.31
S108x4-BC-e90	394.0	1.03	1.32	1.21
S133x4-BC-e0	1858.2	1.18	1.42	1.03
S133x4-BC-e10	1508.4	1.18	1.47	1.11
S133x4-BC-e25	1240.7	1.23	1.57	1.23
S133x4-BC-e25-R	1192.7	1.17	1.50	1.18
S133x4-BC-e55	861.2	1.21	1.61	1.30
S133x4-BC-e110	508.9	1.10	1.50	1.25
S139x6-BC-e0	3107.1	1.31	1.36	1.28
S139x6-BC-e15	2521.1	1.37	1.47	1.45
S139x6-BC-e30	2048.9	1.45	1.60	1.62
S139x6-BC-e60	1357.9	1.35	1.51	1.59
H48.3x5-SBC-e0	331.9	0.93	0.92	1.16
H48.3x5-SBC-e6	234.6	0.96	0.99	1.09
H48.3x5-SBC-e12	189.1	0.96	1.00	1.08
H48.3x5-SBC-e24	139.3	0.95	1.00	1.06
H48.3x5-SBC-e48	92.8	0.93	0.99	1.03
H88.9x5-SBC-e0	910.6	0.99	0.97	1.08
H88.9x5-SBC-e11	664.1	1.01	1.04	1.04
H88.9x5-SBC-e22	534.9	1.03	1.08	1.05
H88.9x5-SBC-e45	384.4	1.03	1.11	1.06
H88.9x5-SBC-e89	242.8	1.01	1.11	1.06
H168.3x4-SBC-e0	1450.1	1.02	1.17	0.95
H168.3x4-SBC-e21	1096.3	1.06	1.27	1.09
H168.3x4-SBC-e42	865.5	1.07	1.31	1.15
H168.3x4-SBC-e84	604.0	1.06	1.33	1.20
H168.3x4-SBC-e168	368.5	1.03	1.32	1.22
H219.1x3-SBC-e0	1197.0	0.95	1.27	0.77
H219.1x3-SBC-e27	931.6	1.00	1.42	0.91
H219.1x3-SBC-e55	724.9	0.97	1.43	0.95
H219.1x3-SBC-e110	498.9	0.94	1.43	0.98
H219.1x3-SBC-e219	308.7	0.91	1.44	1.01
H323.9x3-SBC-e0	1744.4	0.98	1.52	0.75
H323.9x3-SBC-e40	1265.4	1.00	1.64	0.82
H323.9x3-SBC-e81	978.0	1.00	1.68	0.85
H323.9x3-SBC-e162	674.0	1.00	1.73	0.88
H323.9x3-SBC-e324	414.8	0.99	1.76	0.90
V48.3x5-SBC-e0	403.6	0.97	0.98	1.25
V48.3x5-SBC-e6	283.0	0.99	1.03	1.15
V48.3x5-SBC-e12	230.9	0.98	1.03	1.13
V48.3x5-SBC-e24	173.7	0.97	1.03	1.10
V48.3x5-SBC-e48	119.1	0.95	1.02	1.07

V88.9x5-SBC-e0	1244.9	1.04	1.01	1.16
V88.9x5-SBC-e11	883.4	1.06	1.08	1.24
V88.9x5-SBC-e22	706.4	1.08	1.12	1.30
V88.9x5-SBC-e45	507.7	1.09	1.16	1.36
V88.9x5-SBC-e89	324.0	1.08	1.18	1.40
V168.3x4-SBC-e0	1965.4	1.10	1.37	0.97
V168.3x4-SBC-e21	1452.5	1.11	1.45	1.08
V168.3x4-SBC-e42	1131.0	1.10	1.47	1.12
V168.3x4-SBC-e84	781.9	1.07	1.48	1.16
V168.3x4-SBC-e168	475.5	1.02	1.46	1.17
V219.1x3-SBC-e0	1587.3	0.98	1.46	0.76
V219.1x3-SBC-e27	1198.9	1.03	1.58	0.87
V219.1x3-SBC-e55	924.5	1.02	1.59	0.89
V219.1x3-SBC-e110	619.6	0.98	1.55	0.90
V219.1x3-SBC-e219	376.6	0.95	1.54	0.91
V323.9x3-SBC-e0	1887.5	0.99	2.07	0.60
V323.9x3-SBC-e40	1567.4	1.19	2.63	0.75
V323.9x3-SBC-e81	1205.1	1.20	2.71	0.77
V323.9x3-SBC-e162	815.3	1.19	2.77	0.78
V323.9x3-SBC-e324	500.0	1.20	2.85	0.80
S48.3x5-SBC-e0	407.4	0.95	0.96	1.23
S48.3x5-SBC-e6	295.5	1.00	1.04	1.16
S48.3x5-SBC-e12	244.0	1.00	1.05	1.15
S48.3x5-SBC-e24	186.5	0.99	1.05	1.13
S48.3x5-SBC-e48	130.8	0.99	1.06	1.12
S88.9x5-SBC-e0	1423.4	1.10	1.06	1.24
S88.9x5-SBC-e11	996.7	1.12	1.14	1.30
S88.9x5-SBC-e22	796.5	1.14	1.19	1.36
S88.9x5-SBC-e45	573.0	1.15	1.23	1.43
S88.9x5-SBC-e89	368.6	1.15	1.26	1.47
S168.3x4-SBC-e0	2278.9	1.20	1.52	1.03
S168.3x4-SBC-e21	1642.2	1.18	1.58	1.12
S168.3x4-SBC-e42	1275.1	1.15	1.59	1.16
S168.3x4-SBC-e84	873.0	1.11	1.58	1.19
S168.3x4-SBC-e168	523.7	1.05	1.54	1.18
S219.1x3-SBC-e0	1791.4	1.02	1.57	0.79
S219.1x3-SBC-e27	1313.7	1.08	1.73	0.87
S219.1x3-SBC-e55	1007.4	1.07	1.76	0.90
S219.1x3-SBC-e110	684.4	1.06	1.79	0.91
S219.1x3-SBC-e219	404.6	1.02	1.76	0.89
S323.9x3-SBC-e0	2279.5	1.21	2.76	0.66
S323.9x3-SBC-e40	1668.2	1.28	3.07	0.73
S323.9x3-SBC-e81	1283.0	1.29	3.17	0.75

S323.9x3-SBC-e162	875.0	1.30	3.26	0.77
S323.9x3-SBC-e324	532.6	1.30	3.32	0.78
Mean (P_m)		1.08	1.46	1.08
COV (V_P)		0.099	0.352	0.188
Resistance Factor (ϕ)		0.90	0.90	1.00
Reliability Index (β_0)		2.68	2.20	1.83

Table 11. Summary of comparisons of test and FE ultimate capacities with nominal predicted capacities

No. of Tests:40	FE:150	$\frac{P_u}{P_{AISC}}$	$\frac{P_u}{P_{AS}}$	$\frac{P_u}{P_{EC3}}$
LBC: 83	Mean	1.07	1.40	1.04
	COV	0.069	0.362	0.121
SBC: 107	Mean	1.08	1.46	1.08
	COV	0.099	0.352	0.188
ALL: 190	Mean	1.07	1.43	1.07
	COV	0.087	0.356	0.165
	ϕ	0.90	0.90	1.00
	β	2.72	2.14	1.86
	$\phi^{\#}$	0.85	0.85	0.85
	$\beta^{\#}$	2.95	2.28	2.42

#Reliability analysis using resistance factor of 0.85

LBC: Long beam-columns

SBC: Short beam-columns