

Structural response and residual capacity of S700 high strength steel CHS columns after exposure to elevated temperatures

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Abstract: The structural behavior and residual capacity of S700 high strength steel circular hollow section (CHS) columns after exposure to elevated temperatures have been studied through testing and numerical modeling. The testing program was conducted on ten S700 high strength steel CHS columns and comprised heating and cooling of the specimens as well as post-fire material testing, measurements of initial geometric imperfections and pin-ended column tests. Numerical simulations were subsequently performed, where finite element models were built and validated with reference to the experimental results and afterward used to conduct parametric studies to obtain further numerical data. Given that there are no specific provisions for the design of steel structures after exposure to elevated temperatures, the relevant room temperature design buckling curves were evaluated, using post-fire material properties, for their applicability to S700 high strength steel CHS columns after exposure to elevated temperatures, based on the experimental and numerical data. The evaluation results indicated that the buckling curves prescribed in the American Specification and Australian Standard lead

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to accurate and consistent residual capacity predictions for S700 high strength steel CHS columns after exposure to elevated temperatures, while the Eurocode buckling curve yields conservative predictions of residual capacity. A revised Eurocode buckling curve was then proposed and results in a higher level of accuracy than its original counterpart.

Keywords: Circular hollow section; Design analysis; Flexural buckling; Grade S700 high strength steel; Heating and cooling; Numerical modeling; Pin-ended column tests; Post-fire residual capacity

Introduction

High strength steel offers the potential for lighter and more sustainable structures, and is attracting growing interest in both research and practice. The general structural behavior and design of high strength steel tubular elements has, as summarized below for the case of circular hollow section (CHS) members, been the focus of a number of recent studies. Stub column tests have been carried out on hot-finished high strength steel CHS with a material yield strength of 590 MPa (Pournara et al. 2017) and cold-formed high strength steel CHS with material yield strengths ranging from 460 MPa to 1100 MPa (Ma et al. 2016a; Chen et al. 2020; Meng and Gardner 2020b) to examine their cross-sectional behavior and compression resistance. Jiao and Zhao (2004), Ma et al. (2016b) and Meng and Gardner (2020b) experimentally investigated the flexural behavior and resistance of cold-formed high strength steel CHS beams with yield strengths varying from 700 MPa to 1350 MPa. On the basis of the obtained stub column and

beam test data, codified slenderness limits (for classifying cross-sections) and effective section approaches (for treating local buckling) were evaluated and found to yield slightly conservative resistance predictions. Improved design approaches have been developed by Lan et al. (2018, 2019) based on the continuous strength method (Buchanan et al. 2016) and yield more accurate capacity predictions. Experimental investigations into the local stability of cold-formed S700, S900 and S1100 high strength steel CHS under combined loading have been performed by Ma et al. (2017) and Meng and Gardner (2020b), with conservatism in existing codified interaction formulae highlighted. These studies focused on the performance and resistances of high strength steel CHS components at room temperature, while their behavior and residual capacities after exposure to elevated temperatures remain unexplored.

Fire safety is one of the key considerations in the design of steel structures, since, during a fire, exposed structural steel components can heat up rapidly and suffer a degradation in load-carrying capacity. In most scenarios, fire only occurs in parts of a structure and may burn out or be extinguished before it spreads to the entire structure. An accurate evaluation of the extent of damage and the residual load-carrying capacity of structural components after exposure to fire is therefore necessary in order to enable an informed decision to be made as to whether a structure can be directly used again, or should be partially repaired or dismantled. To provide the required underpinning basis to support such a post-fire assessment of structures composed of high strength steel CHS members, experimental and numerical studies into the flexural buckling behavior and residual capacity of S700 high strength steel CHS columns after exposure to elevated temperatures have been conducted and are described in this paper. An

experimental program, including material testing, measurements of initial geometric imperfections as well as pin-ended column tests, was first conducted on two series of S700 high strength steel CHS column specimens after exposure to different levels of elevated temperature. The experimental program was supplemented by numerical simulations, with finite element models were built and validated with reference to the experimental results and afterward used to conduct parametric studies to obtain further numerical data. On the basis of the obtained test and numerical data, the relevant room temperature buckling curves, as given in ANSI/AISC 360-16 (AISC 2016), AS 4100 (Standards Australia 2016) and EN 1993-1-12 (CEN 2007), were examined, using post-fire material properties, for their applicability to assess the capacity of S700 high strength steel CHS columns after exposure to elevated temperatures. Finally, a revised design approach was proposed.

Experimental Program

Overview

In view of the fact that there have been no tests on S700 high strength steel CHS columns after exposure to elevated temperatures, an experimental program was first carried out. Two sizes of CHS – CHS 139.7×4 and CHS 139.7×10 – were adopted, which were cold-rolled and seam-welded from S700 MC high strength steel sheets and respectively categorized as Class 4 and Class 1 at room temperature based on the Eurocode slenderness limits (CEN 2007). The two chosen section sizes therefore allowed the two ends (i.e. Class 1 and Class 4) of the local

geometric slenderness spectrum to be explored. For each CHS, five geometrically identical column specimens were prepared, with four exposed to different nominal elevated temperatures – $T=400\text{ }^{\circ}\text{C}$, $600\text{ }^{\circ}\text{C}$, $900\text{ }^{\circ}\text{C}$ and $1100\text{ }^{\circ}\text{C}$ – and one remaining unheated. Overall, the selected temperatures fell within the range of $30\text{ }^{\circ}\text{C}$ – $1200\text{ }^{\circ}\text{C}$ considered in EN 1993-1-2 (CEN 2005). Furthermore, in view of fact that the post-fire material properties of high strength steels generally remain unchanged for exposure temperatures below $500\text{ }^{\circ}\text{C}$ (Qiang et al. 2012; Li and Young 2018; Su et al. 2020), only one exposure temperature below $500\text{ }^{\circ}\text{C}$ (i.e. $400\text{ }^{\circ}\text{C}$) was selected, while three elevated temperatures above $500\text{ }^{\circ}\text{C}$ (i.e. $600\text{ }^{\circ}\text{C}$, $900\text{ }^{\circ}\text{C}$ and $1100\text{ }^{\circ}\text{C}$ at intervals of $200\text{--}300^{\circ}\text{C}$) were adopted to capture the influence of the exposure temperatures above $500\text{ }^{\circ}\text{C}$ on the post-fire material and column behavior. The labeling system adopted for each column specimen begins with the series identifier ‘S1’ or ‘S2’ (with ‘S1’ signifying the CHS 139.7×4 specimen series and ‘S2’ signifying the CHS 139.7×10 specimen series), followed by a letter ‘T’ along with the nominal target temperature (e.g., ‘T400’ indicates the nominal target temperature of $400\text{ }^{\circ}\text{C}$). For each CHS, five coupon specimens were also cut and heated together with the corresponding column specimens. Overall, the testing program comprised heating and cooling of the specimens as well as post-fire material testing, measurements of initial geometric imperfections and pin-ended column tests.

Heating and Cooling

An electric furnace was employed to heat the coupon and column specimens. As depicted in Fig.1, a series of heating elements are uniformly distributed over both sides of the furnace

chamber, capable of providing stable and uniform elevated temperatures throughout the chamber during the heating process. An embedded temperature probe was adopted to monitor the air temperature in the chamber, while five type-K thermocouples were adhered to the column and coupon specimens at different positions to capture their surface temperatures. The heating rate of the furnace was set to 10 °C/min, which lies within the range of heating rates typically experienced by protected steelwork in fire (Twilt 1988; Wang 2002; Abreu et al. 2020; He et al. 2020). It is also worth noting that the influence of heating rate, up to about 50 °C/min (i.e. the approximate rate of temperature increase for non-insulated steelwork), on the residual material properties of high strength steels has been found to be insignificant (Li and Young 2018; Wang and Lui 2021). Upon reaching the target temperature, the temperature was held constant for a soaking period of about 30 minutes, to ensure a stable and uniform temperature distribution in the column and coupon specimens. Once the heating and soaking processes had been completed, the electric furnace was switched off and all the specimens cooled down naturally to the room temperature. Fig. 2 displays the temperature–time curves measured from the five type-K thermocouples for a typical set of S700 high strength steel CHS column specimens and coupons during the heating, soaking and cooling processes. It can be observed that all the curves are almost identical, revealing that the temperatures at different positions on the specimens are uniform throughout the entire process. The average measured maximum temperature T_m from the five type-K thermocouples for each set of specimens is summarized in Table 1. As shown in Fig. 3, the surface colors of S700 high strength steel became increasingly dark as the exposure temperatures increased, as discussed further by Zhong et al. (2021).

Post-fire Material Testing

Tensile coupon tests were carried out upon heating to obtain their post-fire material properties and stress–strain curves. A Schenck 250 kN universal machine driven in displacement control was employed for all the tensile coupon tests. The displacement rates were set as 0.05 mm/min and 0.80 mm/min (Su et al. 2020; Zhong et al. 2021) before and after the yield stress; the resulting strain rates satisfy the corresponding requirements specified in EN ISO 6892-1 (CEN 2016). The experimental setup is depicted in Fig. 4, which features an extensometer positioned onto the coupon over the central 50 mm to capture the elongations and two strain gauges adhered to both sides of the coupon at midheight to monitor the strains. Figs. 5(a and b) depict the measured post-fire stress–strain curves of the coupons, respectively cut from the CHS 139.7×4 and CHS 139.7×10 profiles. The values of the key measured material properties are summarized in Table 1, including the Young’s modulus E , the yield stress f_y , the ultimate stress f_u , the strain at the ultimate stress ε_u and the strain at fracture ε_f . Note that the yield stress was taken as the lower bound of the yield plateau for the coupons with clearly-defined yield plateaus but the 0.2% proof stress for the coupons without yield plateaus.

Initial Global Geometric Imperfection Measurements

The behavior of high strength steel tubular columns is known to be influenced by initial global geometric imperfections (Wang and Gardner 2017; Meng and Gardner 2020a), measurements of which were thus carried out on the S700 high strength steel CHS column specimens. The

measurement setup is displayed in Fig. 6, showing a column specimen placed onto the table of a CNC router, with an LVDT affixed to the moving router arm. For each column specimen, the imperfection measurement was conducted by moving the LVDT longitudinally along its uppermost edge, with the readings recorded. The magnitude of the initial midheight global geometric imperfection was defined as the deviation from the data point measured at midheight to a reference line connecting the data points measured at the two ends, as illustrated in Fig. 7. Upon completion of the first measurement, the specimen was rotated 60° about the longitudinal axis and the measurement procedure was repeated until the magnitudes of the initial midheight global geometric imperfections had been obtained for all six radial directions, as illustrated in Fig. 6. The largest value measured in all the six radial directions was finally taken as the initial global geometric imperfection magnitude of the specimen ω_g , as summarized in Table 2.

Pin-Ended Column Tests

Having exposed the S700 high strength steel CHS columns to different elevated temperature levels, compression tests were performed to study their post-fire flexural buckling behavior and residual capacity. The measured geometric dimensions of each column specimen, including the outer diameter D , the wall thickness t and the member length L , are summarized in Table 2. Each column specimen was compressed in a displacement-controlled Instron 5000 kN universal machine at a speed of 0.4 mm/min; the resulting strain rate was less than 0.01/s, indicating that quasi-static test conditions were achieved. As displayed in Fig. 8, each end of the machine was equipped with a knife-edge device, which comprises a wedge plate (containing a knife-edge

wedge) and a pit plate (containing a semi-circular groove), to provide pin-ended boundary conditions. Before testing, each column specimen was placed between the top and bottom wedge plates, with its position adjusted to achieve that (i) the longitudinal axis of the specimen was perpendicular to the wedge plates, (ii) the centroid of the specimen end section coincided with the centers of the knife edges and (iii) the radial direction resulting in the maximum magnitude of the initial midheight global geometric imperfection intersected with the knife edges at right angles (He et al. 2020). Upon completion of the member alignment and position adjustment, each column specimen was secured at both ends through stiffening plates that were bolted to the wedge plates. It is worth noting that the distance from the specimen end to the corresponding center of rotation of the knife-edge device was 55 mm (see Fig. 8). Thus, the effective member length is taken as $L_e=L+110$ mm.

The instrumentation used for the pin-ended column tests is displayed in Fig. 8, comprising (i) a pair of LVDTs attached to the specimen at midheight to record the lateral deflections and (ii) two strain gauges adhered to the extreme fibers of the CHS at midheight in the longitudinal direction to record the strains. The readings from the strain gauges and LVDTs were adopted to calculate the actual overall global geometric imperfection magnitude of each column specimen e_m . The formula for calculating e_m is given by Eq. (1) (Zhao et al. 2016; He et al. 2020; Meng and Gardner 2020a; Meng and Gardner 2021). The definition of each parameter adopted in Eq. (1) is as follows: ω_g is the initial midheight global geometric imperfection magnitude, e_0 is the initial end loading eccentricity, I is the second moment of area, $\varepsilon_{max}-\varepsilon_{min}$ is the difference in strain measured from the two strain gauges, Δ is the average lateral deflection

at midheight recorded by the two LVDTs and N is the compression force. In view of the fact that the derivation of Eq. (1) is based on the assumption of linear elastic behavior, the compression forces N used in the calculation of e_m were limited to 15% of the estimated failure loads (He et al. 2020; Meng and Gardner 2021). If the calculated value of e_m was greater than $L_e/1000$, the member position was re-adjusted until the achievement of $e_m < L_e/1000$ (He et al. 2020; Meng and Gardner 2021); note that $L_e/1000$ is the representative value of initial global geometric imperfection generally adopted in the development of design rules for column buckling (Beer and Schulz 1970; Ziemian 2010). Table 3 summarizes the final value of e_m , together with the normalized magnitude e_m/L_e , for each column specimen.

$$e_m = \omega_g + e_0 = \frac{EI(\varepsilon_{max} - \varepsilon_{min})}{DN} - \Delta \quad (1)$$

Figs. 9(a and b) depict the load–midheight lateral deflection curves measured for the CHS 139.7×4 and CHS 139.7×10 column specimens, respectively. The key experimental results are summarized in Table 3, where N_u denotes the failure load and Δ_u denotes the midheight lateral deflection at the failure load. Regarding failure modes, the CHS 139.7×10 column specimens failed by flexural buckling, as illustrated in Fig. 10, while their CHS 139.7×4 counterparts were shown to exhibit local–flexural interactive buckling, as depicted in Fig. 11.

Numerical Modeling Program

Overview

Following the testing program, a numerical modeling program was performed by adopting the finite element (FE) software Abaqus (Abaqus 2014). The FE modeling techniques and procedures are first described. Then, the accuracy of the developed FE models is validated with reference to the experimental results in a validation study. Finally, parametric studies based on the validated FE models are presented, with the aim of deriving a numerical data bank.

Development of FE Models

Each S700 high strength steel CHS column FE model was developed by using the measured geometric dimensions. The shell element S4R (Abaqus 2014), which has been extensively adopted for modeling high strength steel tubular sections (Wang and Gardner 2017; Fang et al. 2018; Meng et al. 2020; Meng and Gardner 2020b; Zhong et al. 2021), was adopted in this numerical modeling program. A mesh sensitivity study, based on eigenvalue buckling analysis results, was carried out to select a suitable element size. Six element sizes, namely $0.01D$, $0.02D$, $0.05D$, $0.1D$, $0.15D$ and $0.2D$, were examined in the mesh sensitivity study. Table 4 reports the results for the CHS 139.7×4 and CHS 139.7×10 column FE models, based on which the element size of $0.05D$ was finally selected, which provided accurate results with good computational efficiency. Regarding the material modeling, a multi-linear elastic-plastic stress–strain model

(Abaqus 2014) was adopted and required the (measured) engineering stress–strain histories to be converted into the true stress–plastic strain responses. Regarding the modeling of the boundary conditions, the end section of each column FE model was first coupled to a concentric reference point, on which the experimental pin-ended boundary conditions were then applied. More specifically, the top reference point was free to rotate about the buckling axis and translate longitudinally, while the bottom reference point was only free to rotate about the same axis. It is also worth highlighting that each reference point was offset by 55 mm longitudinally from the corresponding end section of the column FE model, to accurately reflect the experimental setup and consider the effective member length.

Initial local and global geometric imperfections were incorporated into each column FE model, with the respective distribution patterns assumed to be the lowest elastic local and global buckling mode shapes (Wang and Gardner 2017; Fang et al. 2018; Meng and Gardner 2020b, He et al. 2020). Three global imperfection values – the measured value e_m and two generalized values $L_e/1000$ and $L_e/1500$, together with two local imperfection values – two generalized values $t/10$ and $t/100$, were employed to scale the initial geometric imperfection distribution patterns; this led to a total of six combinations of global and local imperfection values being examined. Finally, the ‘Static, Riks’ (Abaqus 2014) analysis was carried out to solve each FE model and obtain the failure load, load–deformation curve and failure mode.

Validation Study

An assessment of the accuracy of the developed FE models was carried out through comparing the numerical results with the test observations. Table 5 presents the numerical-to-experimental failure load ratios for all the specimens, showing that (i) the best agreement between the experimental and numerical failure loads is achieved by using the measured overall global imperfection magnitude e_m combined with the generalized local imperfection magnitude $t/10$ and (ii) the experimental failure loads are also well predicted when the imperfection magnitude combination of $L_e/1000$ and $t/100$ is adopted. Graphical comparisons between the experimental and numerical load–midheight lateral deflection curves are shown in Fig. 12, in which the initial stiffness, failure load and overall shape of each experimental load–deformation curve are well simulated; note that the numerical load–deformation curves in Fig. 12 were obtained based on the imperfection magnitude combination – $L_e/1000$ and $t/100$. With regard to the failure modes, the flexural buckling of the CHS 139.7×10 column specimens and the local–flexural interactive buckling of the CHS 139.7×4 column specimens are both captured by their numerical counterparts – see Figs. 10 and 11. It may thus be concluded that the FE models can replicate the experimental observations and are thus regarded as having been validated.

Parametric Studies

The validated FE models were used to carry out a series of parametric studies to obtain further numerical results over a wide range of geometric dimensions. In the parametric studies, the

adopted initial local and global imperfection values were respectively taken as $t/100$ and $L_e/1000$. Table 6 presents the cross-section dimensions and effective member lengths of the column FE models. Specifically, the outer cross-section diameters were kept constant at 100 mm and six wall thicknesses from 1.00 mm to 17.32 mm were selected, resulting in the $D/(te^2)$ ratios at room temperature varying from 20 to 120, where $\varepsilon = \sqrt{235 / f_y}$, covering all four classes of cross-section (Class 1–4) defined in EN 1993-1-12 (CEN 2007). For each of the six modeled cross-sections, ten member lengths ranging from 340 mm to 5490 mm (i.e. ten effective lengths ranging from 450 mm to 5600 mm) were chosen, which led to sixty column FE models. Then, five true stress–plastic strain curves, as converted from the measured stress–strain curves of the CHS 139.7×4 profile at room temperature and after exposure to four levels of elevated temperature, were assigned to each column FE model. In total, 300 FE data were finally generated.

Design Analysis

Overview

Upon completion of the testing and numerical modeling program, design analyses were carried out and are reported in this section. In view of the fact that there are no existing design codes for S700 high strength steel CHS columns after exposure to elevated temperatures, the applicability of the relevant room temperature design provisions, as prescribed in ANSI/AISC 360-16 (AISC 2016), AS 4100 (Standards Australia 2016) and EN 1993-1-12 (CEN 2007), was

assessed based on the experimental and numerical data. In the following sub-sections, the room temperature design provisions in each code are first summarized; then, the predicted failure loads $N_{u,pred}$ based on the room temperature design provisions but with the measured post-fire material properties are determined and compared with the experimental and numerical failure loads N_u . All partial safety factors are set equal to 1.0 in the conducted design calculations. Quantitative comparisons, including the mean $N_u/N_{u,pred}$ ratio and the coefficient of variation (COV), are presented in Tables 7–9, whilst graphical comparisons are displayed in Figs. 13–15. Finally, a revised design approach is proposed.

American Specification ANSI/AISC 360-16 (AISC)

The American specification ANSI/AISC 360-16 (AISC 2016) is applicable to the steel design with material yield strengths up to 690 MPa. The AISC design compression resistance of a CHS column $N_{u,AISC}$ can be calculated by the critical stress method, given by Eq. (2),

$$N_{u,AISC} = \begin{cases} A F_{cr} & \text{for } \frac{D}{t} \leq 0.11 \frac{E}{f_y} \\ A_{eff,AISC} F_{cr} & \text{for } 0.11 \frac{E}{f_y} < \frac{D}{t} < 0.45 \frac{E}{f_y} \end{cases} \quad (2)$$

where A is the gross cross-section area, $A_{eff,AISC}$ is the AISC effective cross-section area and can be calculated from Eq. (3), and F_{cr} is the critical stress, as determined from the AISC buckling curve defined by Eq. (4), in which $\bar{\lambda}_{AISC}$ is the AISC member non-dimensional slenderness given by Eq. (5).

$$A_{eff,AISC} = \left[\frac{0.038E}{f_y(D/t)} + \frac{2}{3} \right] A \quad (3)$$

$$F_{cr} = \begin{cases} (0.658^{\bar{\lambda}_{AISC}^2}) f_y & \text{for } \bar{\lambda}_{AISC} \leq 1.5 \\ \frac{0.877}{\bar{\lambda}_{AISC}^2} f_y & \text{for } \bar{\lambda}_{AISC} > 1.5 \end{cases} \quad (4)$$

$$\bar{\lambda}_{AISC} = \sqrt{\frac{Af_y L_e^2}{\pi^2 EI}} \quad (5)$$

The applicability of the relevant AISC room temperature design provisions to S700 high strength steel CHS columns after exposure to elevated temperatures was assessed through comparing the calculated compression resistance predictions $N_{u,AISC}$ with the experimental and numerical failure loads N_u . Fig. 13 displays the normalized experimental and numerical failure loads ($N_u/(Af_y)$ or $N_u/(A_{eff,AISC}f_y)$) plotted against the AISC member non-dimensional slendernesses $\bar{\lambda}_{AISC}$, along with the AISC buckling curve. It is evident from Fig. 13 that the normalized data points for different temperatures exhibit a similar distribution pattern and lie just above the AISC buckling curve. Quantitative comparisons are given in Table 7, where the mean experimental and numerical to AISC predicted failure load ratios $N_u/N_{u,AISC}$ and the corresponding COVs, categorized by cross-section type and temperature, are presented. Both the graphical and quantitative comparisons generally reveal that ANSI/AISC 360-16 (AISC 2016) yields accurate and consistent post-fire failure load predictions for S700 high strength steel CHS columns.

Australian Standard AS 4100 (AS)

The Australian standard AS 4100 (Standards Australia 2016) covers the design of steel structures with yield strengths up to 690 MPa. For CHS columns, the AS design buckling resistance $N_{u,AS}$ is determined as the product of the design cross-section resistance (with Af_y for non-slender CHS and $A_{eff,AS}f_y$ for slender CHS) and the flexural buckling reduction factor α_c – see Eq. (6),

$$N_{u,AS} = \begin{cases} \alpha_c A f_y & \text{for non-slender CHS} \\ \alpha_c A_{eff,AS} f_y & \text{for slender CHS} \end{cases} \quad (6)$$

where $A_{eff,AS}$ is the AS effective cross-section area and calculated based on the corresponding effective outer diameter D_{eff} , as given by Eq. (7), in which $\lambda_e = (D/t)(f_y/250)$ is the section slenderness, and α_c is the flexural buckling reduction factor given by Eq. (8),

$$D_{eff} = \min \left(D \sqrt{\frac{82}{\lambda_e}}, D \left(\frac{246}{\lambda_e} \right)^2, D \right) \quad (7)$$

$$\alpha_c = \xi \left[1 - \sqrt{1 - \left(\frac{90}{\xi \lambda} \right)^2} \right] \leq 1 \quad (8)$$

where λ is the slenderness ratio given by $\lambda = \lambda_n + \alpha_a \alpha_b$, in which λ_n is the modified compression member slenderness given by Eq. (9), in which $\bar{\lambda}_{AS}$ is the AS member non-dimensional slenderness given by Eq. (10), α_a is the compression factor given by Eq. (11), α_b is the compression member section constant and equals to -0.5 for cold-formed CHS members, and ξ is the compression member factor defined by Eq. (12), in which η is the compression member

imperfection factor and determined according to Eq. (13).

$$\lambda_n = \sqrt{\frac{\pi^2 E}{250}} \bar{\lambda}_{AS} \quad (9)$$

$$\bar{\lambda}_{AS} = \begin{cases} \sqrt{\frac{Af_y L_e^2}{\pi^2 EI}} & \text{for non-slender CHS} \\ \sqrt{\frac{A_{eff,AS} f_y L_e^2}{\pi^2 EI}} & \text{for slender CHS} \end{cases} \quad (10)$$

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050} \quad (11)$$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2} \quad (12)$$

$$\eta = 0.00326(\lambda - 13.5) \geq 0 \quad (13)$$

The AS room temperature design formulae were assessed for their applicability to S700 high strength steel CHS columns after exposure to elevated temperatures. The test and numerical failure loads, normalized by the AS cross-section resistances and categorized by temperature, are plotted against the AS non-dimensional member slenderness in Fig. 14, together with the AS buckling curve defined by Eq. (8). The AS buckling curve is shown to offer a good representation of all the normalized data points for S700 high strength steel CHS columns after exposure to different levels of elevated temperature. The accuracy and consistency of the AS buckling curve can also be seen in Table 8, where the quantitative evaluation results are presented.

Eurocode EN 1993-1-12 (EC3)

The Eurocode EN 1993-1-12 (CEN 2007) is applicable to high strength steel members with material grades greater than S460 up to S700. Regarding the design of high strength steel CHS columns, EN 1993-1-12 (CEN 2007) employs the same approach as that specified in prEN 1993-1-1:2020 (CEN 2020) for normal strength steel CHS columns. The corresponding design buckling resistance $N_{u,EC3}$ is given by Eq. (14),

$$N_{u,EC3} = \begin{cases} \chi A f_y & \text{for Class 1, 2 and 3 CHS} \\ \chi A_{eff,EC3} f_y & \text{for Class 4 CHS} \end{cases} \quad (14)$$

where $A f_y$ is the cross-section compression resistance for Class 1, 2 and 3 CHS, $A_{eff,EC3} f_y$ is the cross-section compression resistance for Class 4 CHS, in which $A_{eff,EC3}$ is the EC3 effective cross-section area (Chan and Gardner 2008; Chan et al 2010) and can be calculated from Eq. (15), and χ is the reduction factor for column buckling, defined by Eq. (16),

$$A_{eff,EC3} = A \sqrt{\frac{90 \varepsilon^2}{D/t}} \quad \text{for } D/t \leq 240 \varepsilon^2 \quad (15)$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_{EC3}^2}} \quad (16)$$

where $\bar{\lambda}_{EC3}$ is the EC3 non-dimensional member slenderness, as defined by Eq. (17), and ϕ is a buckling parameter, as determined from Eq. (18), in which α is the imperfection factor considering the influences of residual stresses and initial geometric imperfections on column capacity; for cold-formed CHS columns, buckling curve ‘c’ is adopted with $\alpha=0.49$.

$$\bar{\lambda}_{EC3} = \begin{cases} \sqrt{\frac{Af_y L_e^2}{\pi^2 EI}} & \text{for Class 1, 2 and 3 CHS} \\ \sqrt{\frac{A_{eff,EC3} f_y L_e^2}{\pi^2 EI}} & \text{for Class 4 CHS} \end{cases} \quad (17)$$

$$\phi = 0.5[1 + \alpha(\bar{\lambda}_{EC3} - 0.2) + \bar{\lambda}_{EC3}^2] \quad (18)$$

The EC3 resistance predictions for S700 high strength steel CHS columns after exposure to elevated temperatures were assessed from Eqs. (15)–(18) and then compared with the experimental and numerical failure loads. The mean ratios of the experimental and numerical to EC3 predicted failure loads, together with the COVs, are presented in Table 9, while the normalized experimental and numerical failure loads are compared with EC3 buckling curve ‘c’ in Fig. 15, showing that EN 1993-1-12 (CEN 2007) provides safe-sided, but excessively conservative, failure load predictions for S700 high strength steel CHS columns after exposure to elevated temperatures.

Revised EC3 Design Approach (EC3*)

Given that the EC3 buckling curve ‘c’ lies well below the data points in Fig. 15 and leads to overly-conservative buckling resistance predictions, a revised EC3 buckling curve is proposed herein in order to offer a better representation of the data points. On the basis of the general distribution of the normalized experimental and numerical data points, it is proposed to use the EC3 buckling curve ‘a’ with $\alpha=0.21$ in place of the original EC3 buckling curve ‘c’ for cold-formed high strength steel CHS columns after fire exposure; buckling curve ‘a’ is currently

applied to hot-finished steel CHS column in prEN 1993-1-1:2020 (CEN 2020). It is worth noting that one of the effects of exposure to elevated temperatures is to relieve, at least partially, residual stress, in particular the high through-thickness bending residual stresses in the case of cold-formed steel tubes (Gardner et al. 2010; Jandera et al. 2008). The effect of this can be seen from the shape of the measured material stress–strain curves in Fig. 5. More specifically, the stress–strain curves of the coupons that have not been exposed to elevated temperatures show a rounded response, owing to the influence of the through-thickness residual stresses (Gardner and Yun 2018). However, those that have been exposed to elevated temperatures exhibit a sharply-defined yield point, akin to that displayed by hot-rolled steels (Yun and Gardner 2017), and a yield plateau of increasing length with increasing exposure temperature. The fact that the stability response of cold-formed steel tubular columns after exposure to elevated temperatures is similar to that of hot-finished tubular columns would therefore be anticipated. Indeed, shown in Fig. 16, buckling curve ‘a’ provides a good match to the trend of all the data points across the full member non-dimensional slenderness range. A quantitative assessment of the use of EC3 buckling curve ‘a’ was carried out through comparing the predicted buckling resistances $N_{u,EC3*}$ with the experimental and numerical failure loads N_u , as reported in Table 10. It is evident that EC3 buckling curve ‘a’ yields much more accurate and consistent post-fire resistance predictions for S700 high strength steel CHS columns than the original buckling curve ‘c’.

Conclusions

The post-fire flexural buckling behavior and residual capacity of S700 high strength steel CHS columns have been studied through testing and numerical modeling. The testing program was first carried out on ten S700 high strength steel CHS columns after exposure to a range of elevated temperature levels, with the test procedures, setups and results reported in detail. The test results were then employed in a numerical modeling program for validating FE models, which were afterward used to perform a series of parametric studies to obtain additional numerical data over a wide range of geometric dimensions. The obtained experimental and numerical data were adopted to evaluate the applicability of the relevant room temperature design provisions for high strength steel CHS columns, as prescribed in ANSI/AISC 360-16 (AISC 2016), AS 4100 (Standards Australia 2016) and EN 1993-1-12 (CEN 2007), to S700 high strength steel CHS columns after exposure to elevated temperatures. The evaluation results generally showed that the AISC and AS buckling curves offer a good level of design accuracy and consistency when used for S700 high strength steel CHS columns after exposure to elevated temperatures, while the EC3 buckling curve 'c' provides rather conservative residual resistance predictions. It was recommended that the EC3 buckling curve 'a' be adopted for S700 high strength steel CHS columns after exposure to elevated temperatures, reflecting the residual stress-relieving effect. The EC3 buckling curve 'a' resulted in much more accurate and consistent predicted resistances than the original EC3 buckling curve 'c'.

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Data Availability Statement

All data, code and models generated and used during this study appear in the submitted article.

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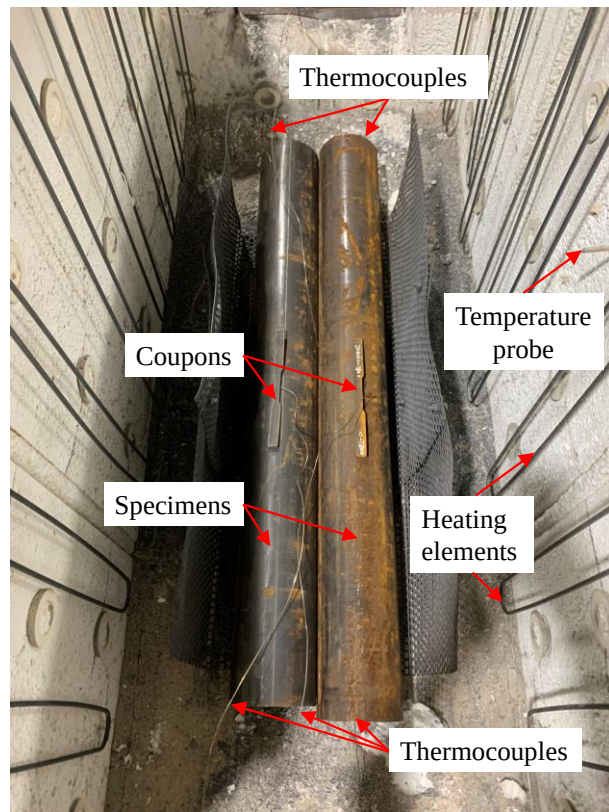


Fig. 1. Electric furnace.

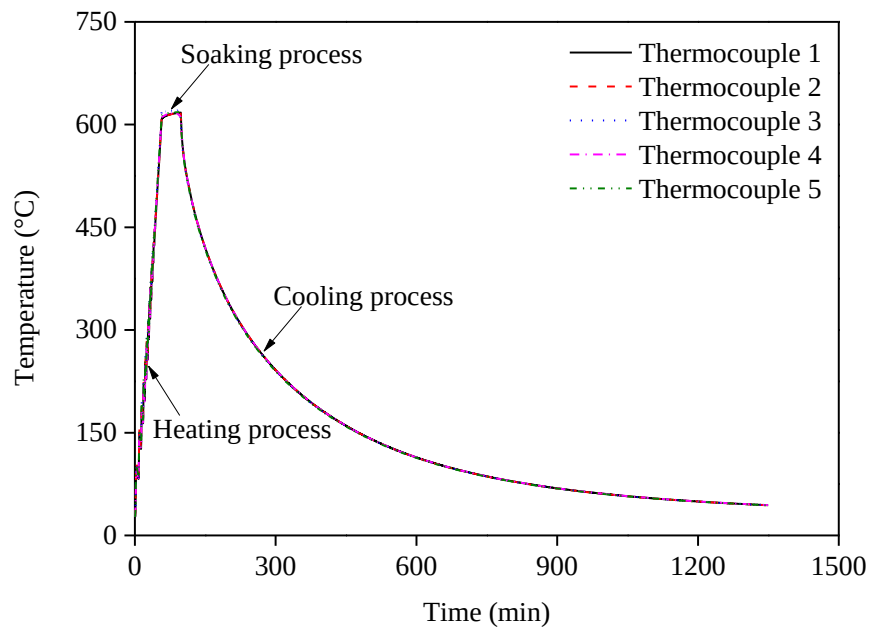


Fig. 2. Full temperature–time curves for a typical set of column specimens and coupons.



Fig. 3. Post-fire surface colors of S700 high strength steel.

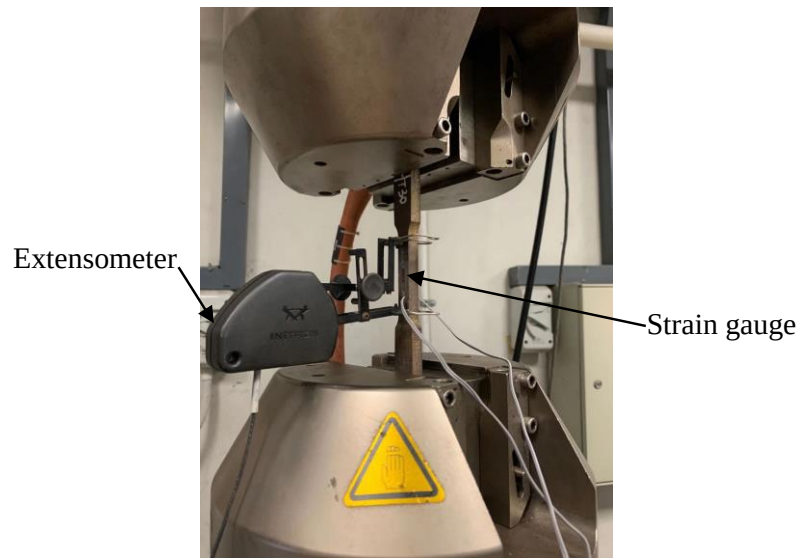


Fig. 4. Setup for coupon tests.

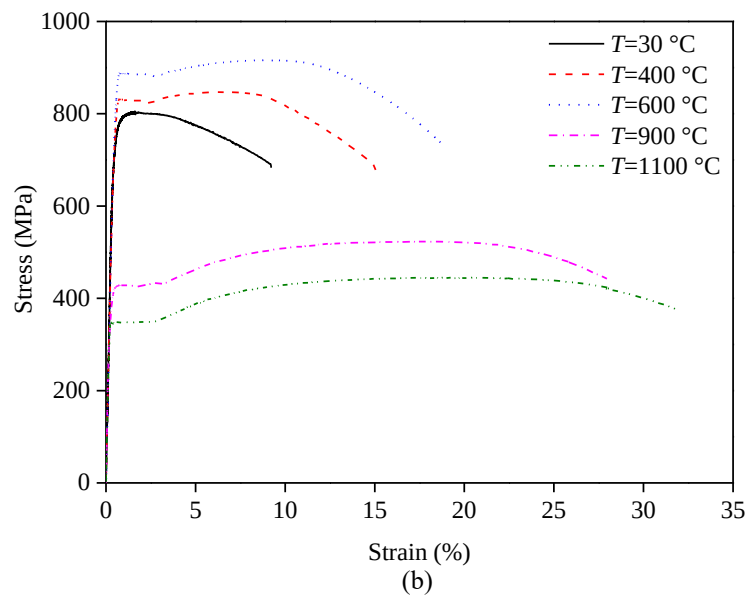
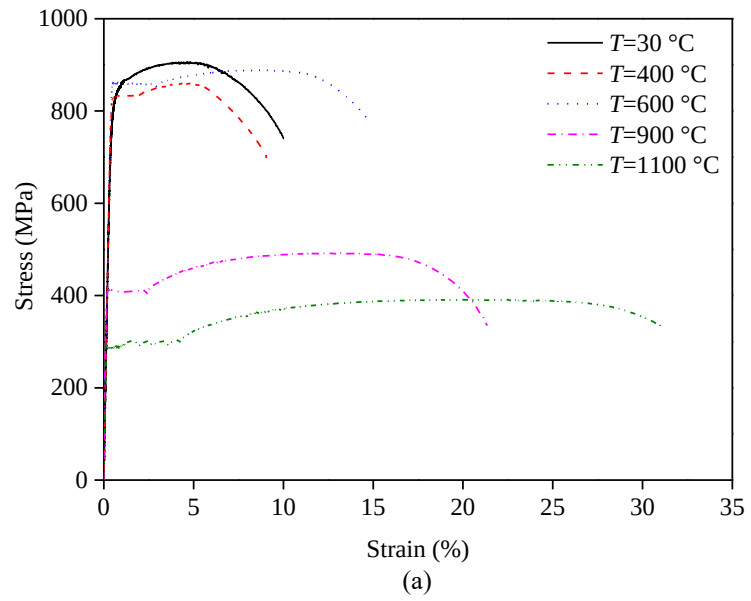


Fig. 5. Measured post-fire stress-strain curves: (a) CHS 139.7x4; and (b) CHS 139.7x10.

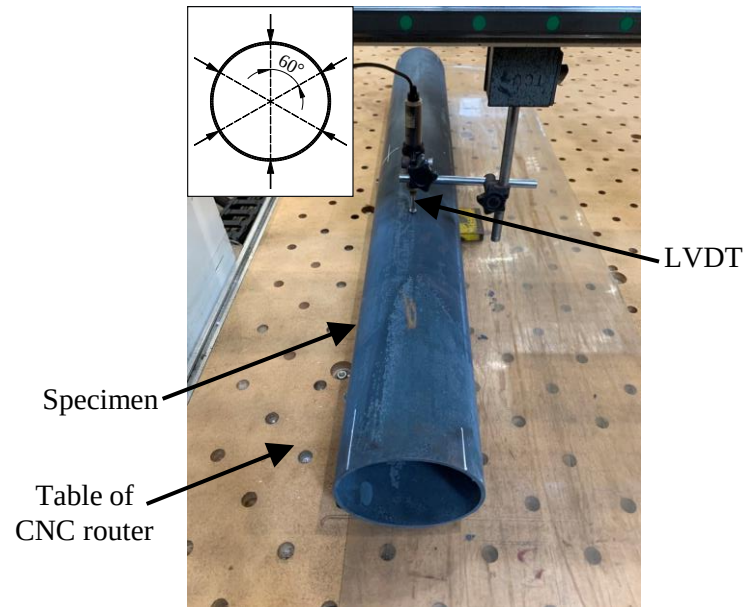


Fig. 6. Measurement setup for initial global geometric imperfections.

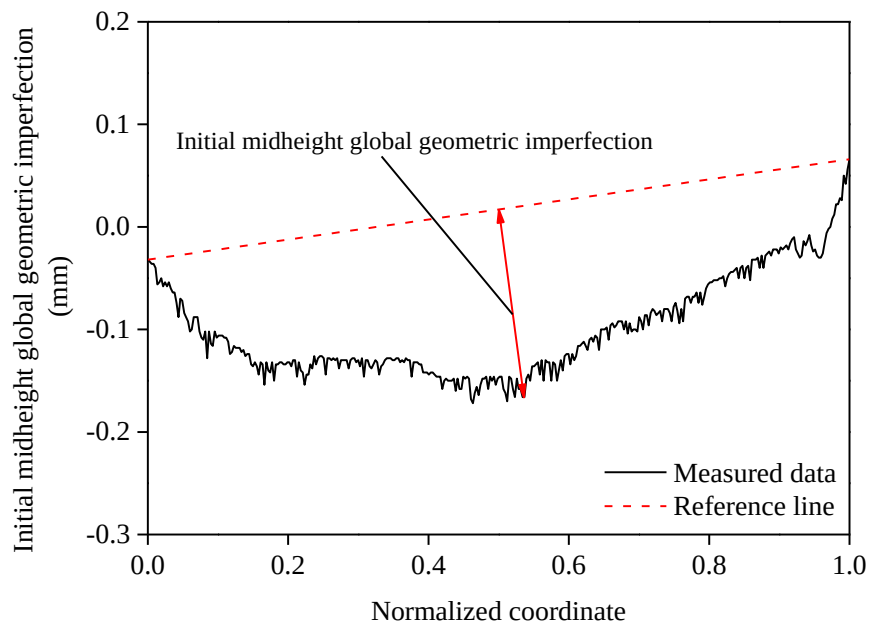


Fig. 7. Determination of initial midheight global geometric imperfection magnitude.

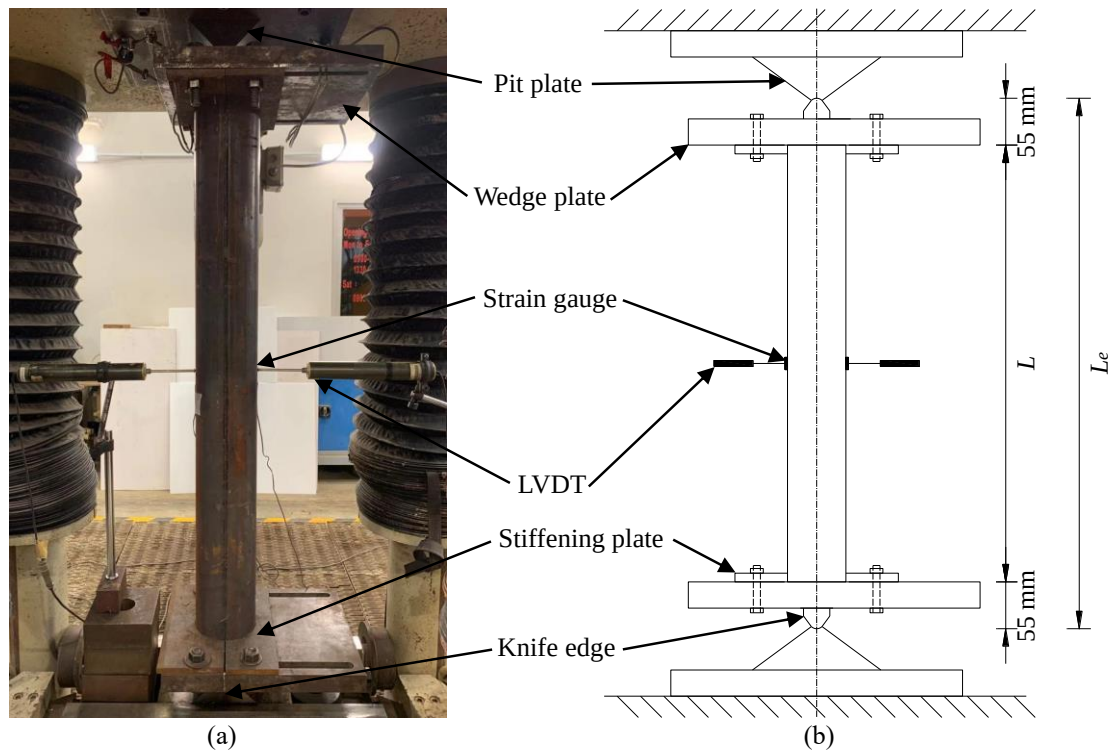


Fig. 8. Pin-ended column test setup: (a) photograph; and (b) schematic diagram.

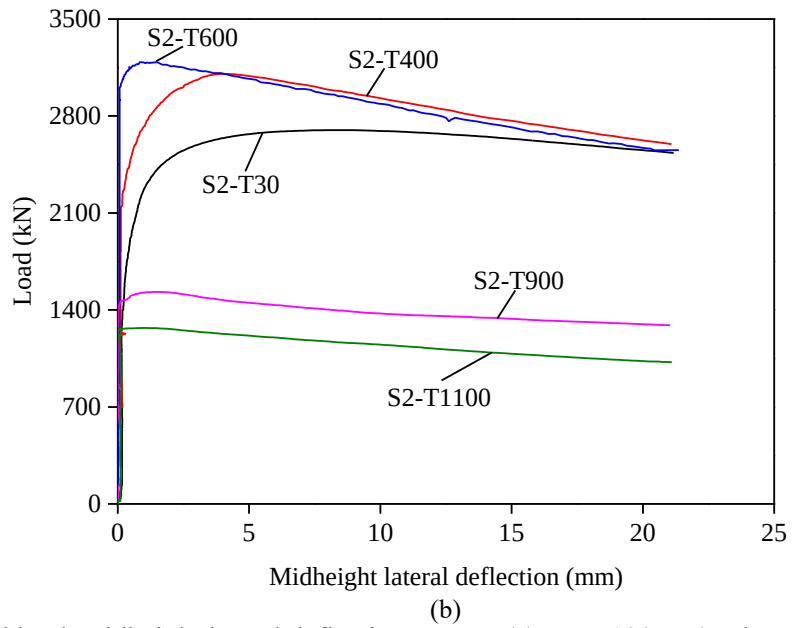
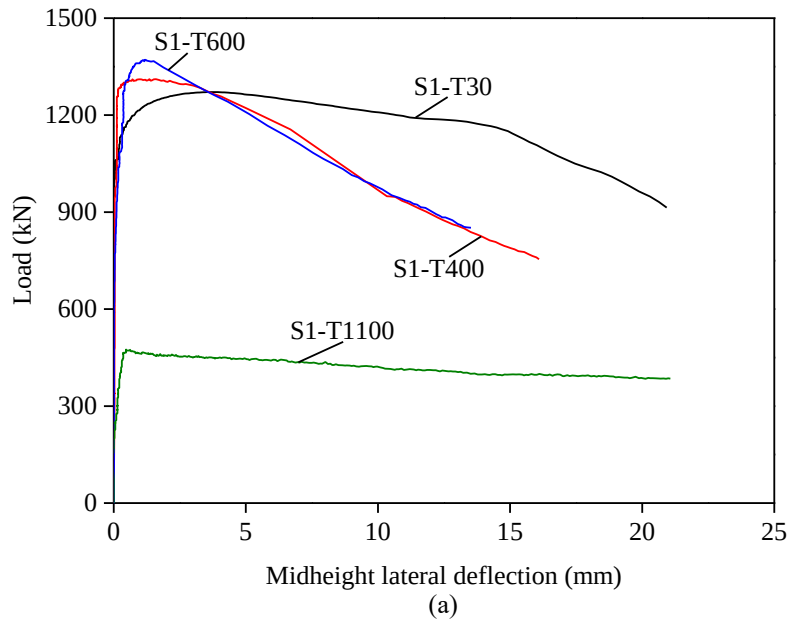


Fig. 9. Measured load-midheight lateral deflection curves: (a) CHS 139.7x4 column specimens; and (b) CHS 139.7x10 column specimens.

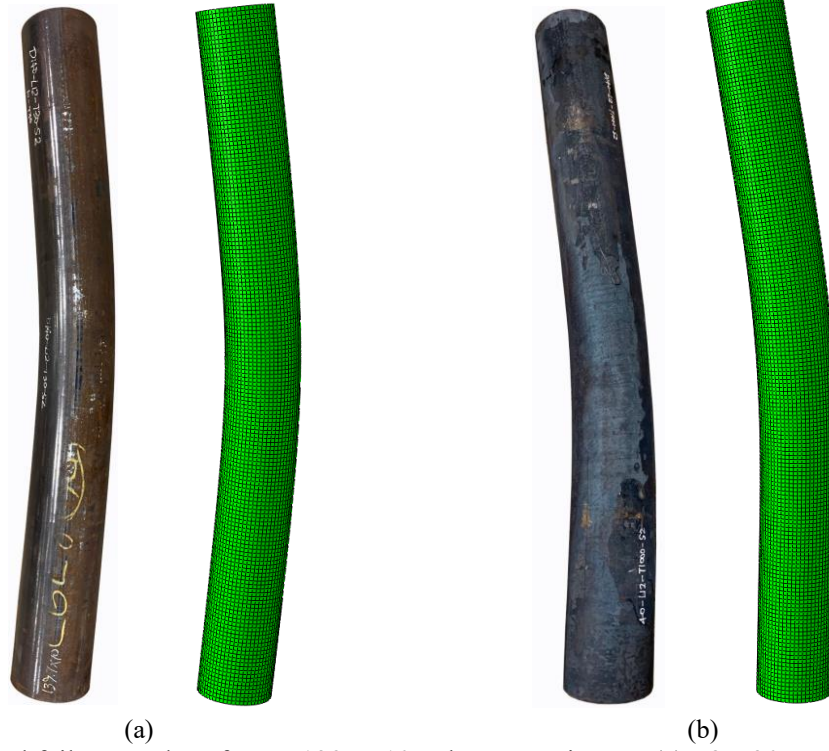


Fig. 10. Typical failure modes of CHS 139.7×10 column specimens: (a) S2-T30; and (b) S2-T1100.

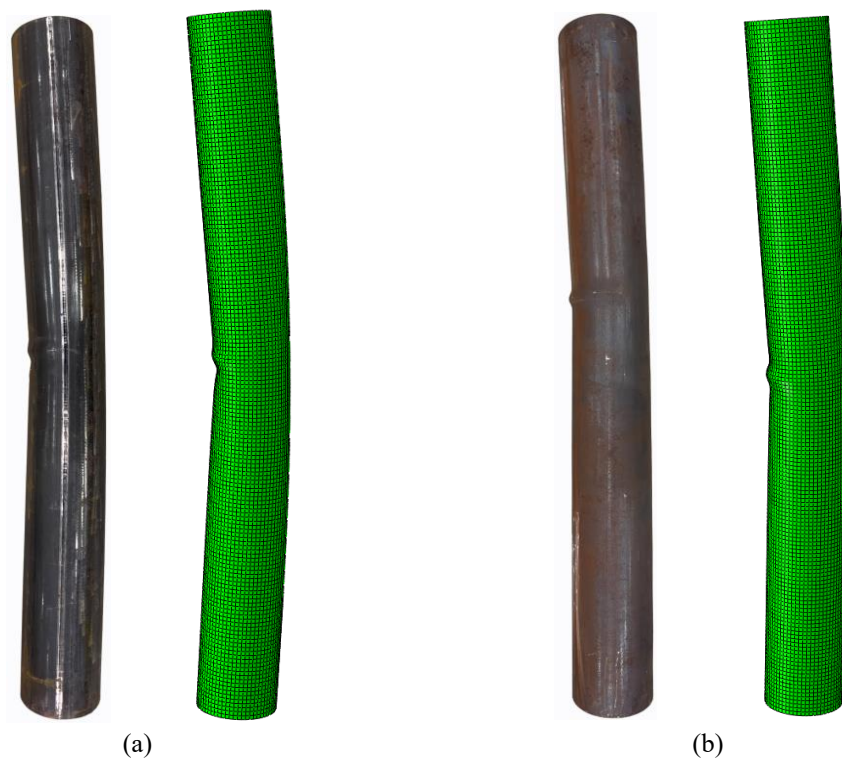


Fig. 11. Typical failure modes of CHS 139.7×4 column specimens: (a) S1-T30; and (b) S1-T600.

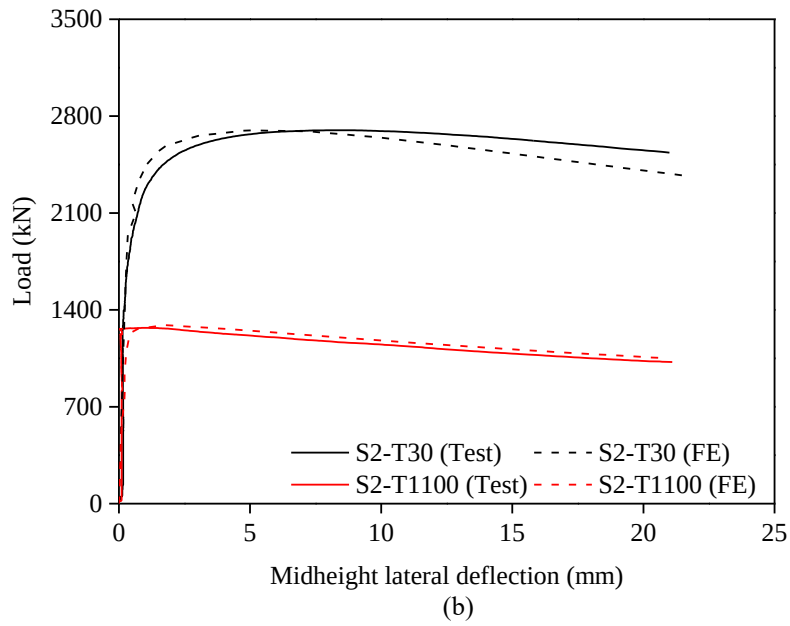
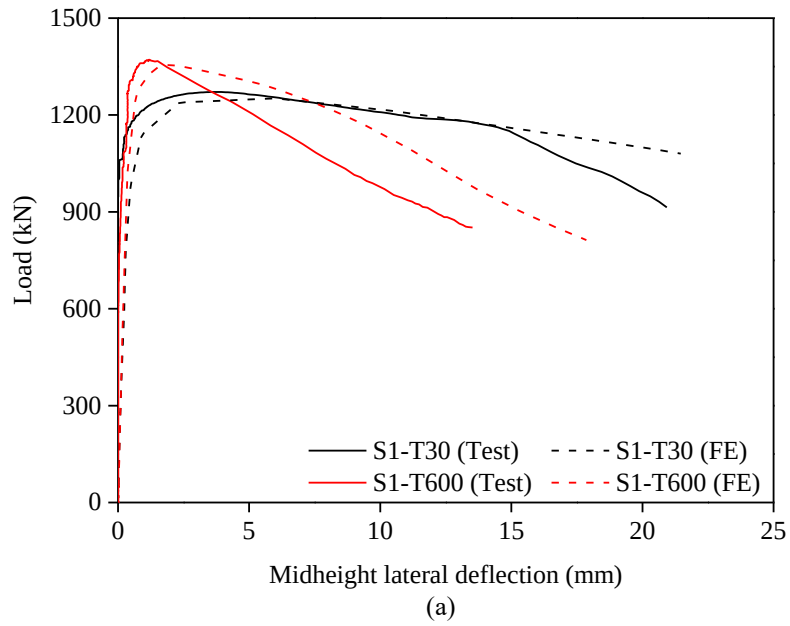


Fig. 12. Comparison between experimental and numerical load–midheight lateral deflection curves for typical column specimens: (a) S1-T30 and S1-T600; and (b) S2-T30 and S2-T1100.

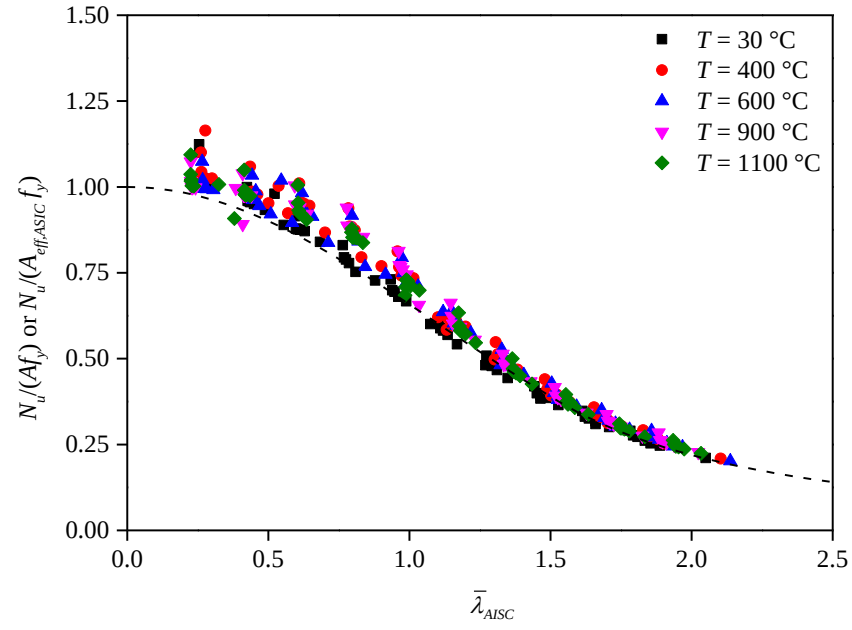


Fig. 13. Comparisons of experimental and numerical failure loads with AISC buckling curve.

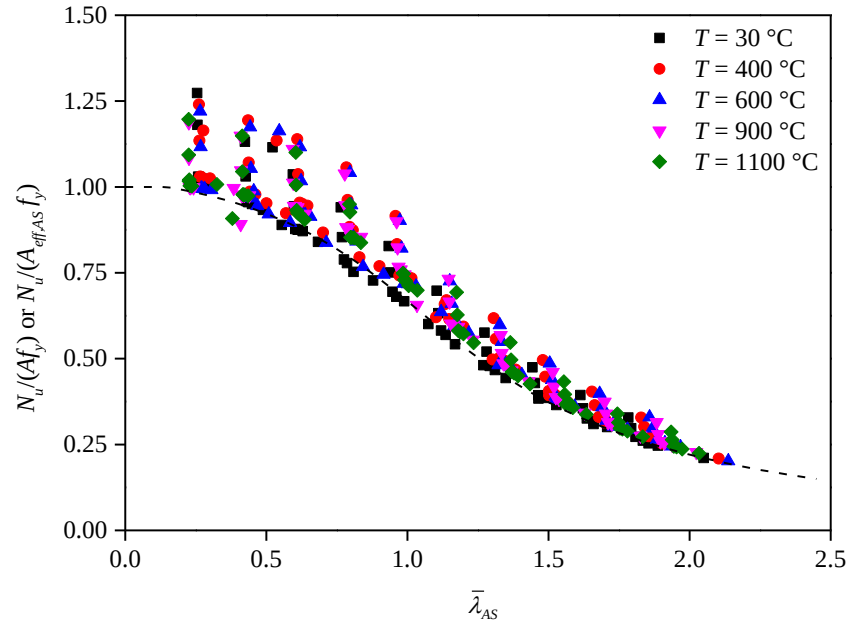


Fig. 14. Comparisons of experimental and numerical failure loads with AS buckling curve.

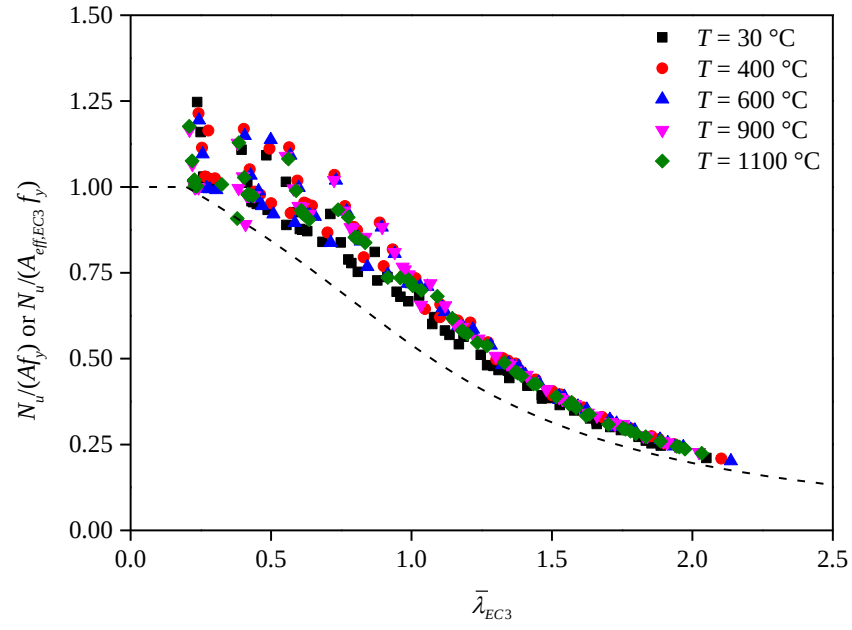


Fig. 15. Comparisons of experimental and numerical failure loads with EC3 buckling curve.

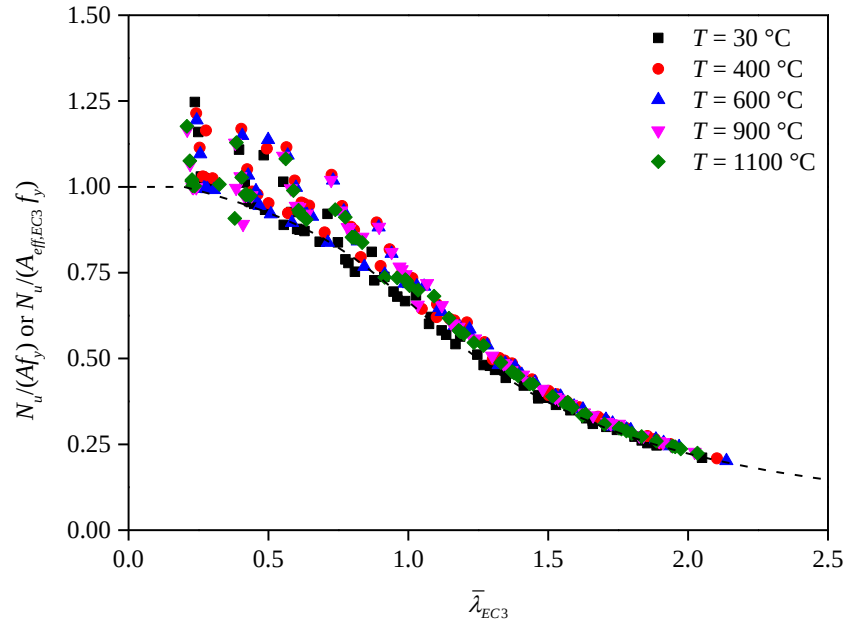


Fig. 16. Comparison of experimental and numerical failure loads with revised EC3 buckling curve.

Table 1 Measured material properties.

Cross-section	T_m (°C)	E (MPa)	f_y (MPa)	f_u (MPa)	ε_u (%)	ε_f (%)
CHS 139.7×4	30	218300	814.2	906.7	4.8	12.0
	395	211700	830.9	861.0	4.7	12.0
	620	212000	859.0	889.0	8.6	20.0
	883	203300	408.4	492.4	12.9	25.0
	1095	196000	282.8	391.7	21.3	32.0
CHS 139.7×10	30	203100	762.0	804.9	1.6	16.0
	395	210400	831.0	848.0	6.4	23.0
	620	213100	887.2	916.6	8.8	25.0
	883	216000	427.0	523.6	16.9	32.0
	1095	196300	342.6	445.8	19.5	41.0

Table 2 Measured geometric dimension and imperfections for S700 high strength steel CHS column specimens.

Cross-section	Specimen ID	D (mm)	t (mm)	L (mm)	L_e (mm)	ω_g (mm)
CHS 139.7×4	S1-T30	139.15	3.93	1200	1310	0.06
	S1-T400	139.05	3.94	1201	1311	0.08
	S1-T600	139.30	3.95	1204	1314	0.39
	S1-T900	139.20	3.93	1205	1315	0.28
	S1-T1100	139.25	3.92	1200	1310	0.13
CHS 139.7×10	S2-T30	139.53	9.77	1199	1309	0.23
	S2-T400	139.50	9.88	1200	1310	0.16
	S2-T600	139.58	9.85	1199	1309	0.10
	S2-T900	139.05	9.91	1204	1314	0.28
	S2-T1100	139.35	9.88	1200	1310	0.19

Table 3 Key pin-ended column test results.

Cross-section	Specimen ID	T_m (°C)	e_m (mm)	e_m/L_e	N_u (kN)	Δ_u (mm)
CHS 139.7×4	S1-T30	30	1.23	1/1065	1271.5	3.69
	S1-T400	395	0.73	1/1796	1311.4	1.35
	S1-T600	620	1.10	1/1195	1371.3	1.18
	S1-T900	883	1.19	1/1105	679.7	*
	S1-T1100	1095	1.02	1/1284	474.7	0.47
CHS 139.7×10	S2-T30	30	1.15	1/1138	2698.0	8.65
	S2-T400	395	1.28	1/1023	3087.5	4.12
	S2-T600	620	1.02	1/1284	3189.0	1.50
	S2-T900	883	0.85	1/1545	1530.4	1.50
	S2-T1100	1095	1.22	1/1074	1250.1	0.97

*LVDTs dropped during testing.

Table 4 Mesh sensitivity study results: (a) CHS 139.7×4 column FE models; and (b) CHS 139.7×10 column FE models.

(a)

Element size (mm)	$0.01D$	$0.02D$	$0.05D$	$0.1D$	$0.15D$	$0.2D$
Elastic buckling load (kN)	4807.4	4805.5	4801.1	4749.4	4676.8	4604.6
Number of elements	258264	64144	10416	2520	1120	672
Times of computation (s)	1300	101	35	28	25	26

(b)

Element size (mm)	$0.01D$	$0.02D$	$0.05D$	$0.1D$	$0.15D$	$0.2D$
Elastic buckling load (kN)	9465.3	9470.4	9472.5	9393.5	9260.5	8995.4
Number of elements	250828	62780	9976	2580	1160	602
Time of computation (s)	304	85	32	24	24	23

Table 5 Comparisons of test failure loads with FE failure loads.

Cross-section	Specimen ID	FE N_u /Test N_u					
		$e_m+t/100$	$L_e/1000+t/100$	$L_e/1500+t/100$	$e_m+t/10$	$L_e/1000+t/10$	$L_e/1500+t/10$
CHS 139.7×4	S1-T30	0.981	0.985	0.996	0.981	0.984	0.996
	S1-T400	1.054	1.028	1.050	1.053	1.027	1.048
	S1-T600	1.001	0.988	1.009	0.996	0.987	1.007
	S1-T900	0.988	0.986	0.996	0.986	0.984	0.993
	S1-T1100	0.991	0.985	0.994	0.989	0.984	0.992
CHS 139.7×10	S2-T30	1.004	1.000	1.014	1.002	0.997	1.010
	S2-T400	0.953	0.957	0.974	0.950	0.949	0.957
	S2-T600	0.994	0.984	1.000	0.986	0.976	0.985
	S2-T900	1.013	0.997	1.013	1.006	0.990	1.003
	S2-T1100	1.058	1.057	1.066	1.045	1.043	1.052
Mean		1.004	0.997	1.011	0.999	0.992	1.004
COV		0.032	0.028	0.027	0.030	0.026	0.028

Table 6 Detailed dimensions of column FE models in parametric studies.

T (°C)	D (mm)	$D/(t\epsilon^2)$	L_e (mm)
30	100	20, 40, 60, 80, 100, 120	450, 750, 1050, 1350, 1650, 1950, 2250, 2550, 2850, 3150
400	100	20, 40, 60, 80, 100, 120	450, 750, 1050, 1350, 1650, 1950, 2250, 2550, 2850, 3150
600	100	20, 40, 60, 80, 100, 120	450, 750, 1050, 1350, 1650, 1950, 2250, 2550, 2850, 3150
900	100	20, 40, 60, 80, 100, 120	550, 1000, 1450, 1900, 2350, 2800, 3250, 3700, 4150, 4600
1100	100	20, 40, 60, 80, 100, 120	650, 1200, 1750, 2300, 2850, 3400, 3950, 4500, 5050, 5600

Table 7 Quantitative assessment of ANSI/AISC 360-16.

T (°C)	$N_u/N_{u,AISC}$					
	Non-slender CHS: 217		Slender CHS: 93		Total: 310	
	Mean	COV	Mean	COV	Mean	COV
30	1.00	0.04	1.06	0.04	1.01	0.04
400	1.07	0.04	1.11	0.05	1.09	0.04
600	1.06	0.03	1.11	0.04	1.07	0.04
900	1.06	0.04	1.10	0.05	1.07	0.05
1100	1.05	0.03	1.09	0.04	1.06	0.04
Total	1.05	0.04	1.10	0.05	1.06	0.05

Table 8 Quantitative assessment of AS 4100.

T (°C)	$N_u/N_{u,AS}$					
	Non-slender CHS: 207		Slender CHS: 103		Total: 310	
	Mean	COV	Mean	COV	Mean	COV
30	1.02	0.02	1.10	0.06	1.05	0.06
400	1.08	0.03	1.16	0.06	1.11	0.06
600	1.06	0.03	1.16	0.05	1.09	0.06
900	1.04	0.03	1.12	0.06	1.07	0.05
1100	1.02	0.03	1.07	0.07	1.03	0.05
Total	1.04	0.04	1.12	0.07	1.07	0.06

Table 9 Quantitative assessment of EN 1993-1-12.

T (°C)	$N_u/N_{u,EC3}$					
	Non-slender CHS: 207		Slender CHS: 103		Total: 310	
	Mean	COV	Mean	COV	Mean	COV
30	1.16	0.05	1.24	0.04	1.18	0.05
400	1.24	0.07	1.32	0.07	1.27	0.07
600	1.22	0.08	1.32	0.07	1.25	0.08
900	1.22	0.09	1.30	0.08	1.24	0.09
1100	1.21	0.08	1.26	0.06	1.23	0.08
Total	1.21	0.08	1.29	0.07	1.23	0.08

Table 10 Quantitative assessment of revised EN 1993-1-12.

T (°C)	$N_u/N_{u,EC3}$ *					
	Non-slender CHS: 207		Slender CHS: 103		Total: 310	
	Mean	COV	Mean	COV	Mean	COV
30	1.00	0.02	1.07	0.06	1.02	0.05
400	1.07	0.03	1.14	0.05	1.10	0.05
600	1.05	0.03	1.14	0.05	1.08	0.05
900	1.06	0.03	1.12	0.05	1.08	0.05
1100	1.05	0.06	1.09	0.05	1.06	0.04
Total	1.05	0.037	1.11	0.06	1.07	0.05