Performance of concrete-filled stainless steel tubular (CFSST)
columns after exposure to fire

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ABSTRACT

The post-fire performance of concrete-filled stainless steel tubular (CFSST) columns subjected to an entire loading–fire history, including four characteristic phases: (i) ambient temperature loading, (ii) heating, (iii) cooling with constant external loads, and (iv) post-fire loading, is investigated in this paper. Sequentially coupled thermal-stress analyses are performed using ABAQUS to establish the temperature field and structural response of CFSST columns. To improve the precision of the finite element analysis (FEA) models, the influence of moisture on the thermal conductivity and specific heat of the concrete in the heating and cooling phases is considered by using subroutines. Existing fire and post-fire test data on CFSST columns are used to validate the FEA modelling. Comparisons between FEA and test results indicate that the accuracy of the model is acceptable; the FEA model is then extended to simulate CFSST columns subjected to the four characteristic phases. The behaviour of the CFSST columns during the four characteristic phases is explained by analysis of the temperature distribution, load versus axial deformation relations, failure modes and internal force redistribution. The excellent post-fire performance of CFSST columns is examined in comparison with traditional concrete-filled carbon steel tubular (CFST) columns with the same total cross-sectional area. The residual strength index is studied with respect to a series of parametric analyses. It is found that the residual strength of CFSST columns is higher than that of CFST columns after the same fire exposure, and that the diameter of the stainless steel tube, slenderness, heating time ratio and load ratio have a significant influence on the residual strength index.

KEYWORDS

Concrete-filled stainless steel tubular (CFSST) columns, Post-fire, Finite element analysis (FEA) modelling, Residual strength, Fire safety engineering

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**Nomenclature**

- $A_c$: Cross-sectional area of concrete
- $A_s$: Cross-sectional area of steel tube
- $c_c$: Specific heat of concrete
- $D$: Diameter of stainless steel tube
- $e$: Load eccentricity
- $e/r$: Load eccentricity ratio
- $r$: Radius of stainless steel tube ($=D/2$)
- $f_{cu}$: Concrete cube strength at ambient temperature
- $f_y$: Yield strength of steel at ambient temperature
- $H$: Height of column
- $k_r$: Residual strength index of column
- $N$: Load
- $N_F$: External load applied to the column
- $N_u$: Ultimate load-carrying capacity at ambient temperature
- $N_{ur}$: Residual strength of column after fire
- $n$: Column load ratio, $n = N_F/N_u$
- $p_c$: Proportion of load supported by concrete
- $p_t$: Proportion of load supported by steel tube
- $T$: Temperature, in °C
- $T_{max}$: Peak temperature attained by material, in °C
- $T_o$: Ambient temperature, in °C
- $t$: Time
- $t_R$: Fire resistance time
- $t_h$: Heating time
- $t_o$: Heating time ratio ($=t_h/t_R$)
- $\Delta$: Axial deformation
- $\alpha$: Steel ratio, $\alpha = A_s/A_c$
- $\varepsilon$: Strain
- $\lambda$: Slenderness ratio ($=4H/D$)
- $\lambda_c$: Thermal conductivity of concrete
- $\sigma$: Stress
- $\sigma_{0.2}$: 0.2% proof (yield) strength of stainless steel at ambient temperature
- $\sigma_u$: Ultimate strength of stainless steel at ambient temperature
1. Introduction

By replacing the outer carbon steel tube with a stainless steel tube, concrete-filled stainless steel tubular (CFSST) columns can combine the advantages of traditional concrete-filled carbon steel tubular (CFST) columns and stainless steel, resulting in greater corrosion resistance, enhanced ductility and improved fire resistance [1, 2]. CFSST columns therefore have great potential for application in engineering structures and facilities with high durability and ductility demands, such as offshore engineering structures.

Over the past ten years, there have been a number of studies into the static response of CFSST columns. Experimental and numerical investigations have been conducted on CFSST stub columns without stiffeners [3–7], with stiffeners [8–10], and involving lean duplex stainless steel [11] and elliptical cross-sections [12, 13] under axial compression; CFSST columns subjected to combined axial compression and bending have been studied by Uy et al. [7]. The results show that CFSST columns generally have improved structural performance over traditional CFST columns and that the existing design codes for CFST columns are somewhat conservative in predicting the load-carrying capacities of both stub and slender CFSST columns.

Fire is recognised as a significant hazard during the life-cycle of a structure, and it is a regulatory requirement of any building design to maintain structural integrity during fire attack. Recently, some research has been focussed on the fire resistance of CFSST columns. Han et al. [14] tested five axially-loaded CFSST columns at elevated temperature, and the tested parameters included section type (circular or square), load ratio (0.15–0.45) and outer section dimension (300–630 mm). The test results were then used to validate finite element analysis (FEA) models. Finally, a comprehensive parametric analysis was conducted and the parametric analysis showed that the section type, section size, slenderness ratio and load ratio are critical parameters. Tao and Ghannam [15] employed FEA modelling to determine the temperature field in CFST and CFSST columns. A comparison between the two types of column showed that lower temperature development is found in the CFSST columns as a result of the lower thermal conductivity and emissivity of stainless steel.
Tao et al. [16] conducted an experimental and analytical investigation into CFSST columns in fire and after fire exposure. A total of 12 specimens were tested, including six CFSST columns in fire, while the remaining six CFSST columns were subjected to sequential ambient temperature loading, fire exposure including both heating and cooling phases under constant applied load and post-fire loading phases. The test results showed that CFSST columns have excellent fire performance and that internal steel rebars can increase both the fire resistance and the post-fire residual strength of CFSST columns. An FEA model was also established and shown to be able to replicate the experimental results of the fire resistance of CFSST columns with reasonable accuracy. However, only the heating phase was simulated for the post-fire test specimens. Chen et al. [17] conducted experimental research on axially loaded square CFSST stub columns after exposure to constant high temperature, and four parameters, including temperature, wall thickness, concrete strength and cooling method, were examined. A strength reduction coefficient for square CFSST stub columns after constant high temperature exposure was also proposed. Tan et al. [18, 19] performed FEA of the fire behaviour of steel reinforced CFSST columns with outer circular and square cross-sections; parametric analyses indicated that, with respect to fire resistance, optimized steel reinforced CFSST columns were superior to CFST and CFSST columns with the same total cross-sectional area of steel or the equivalent cross-sectional load-bearing capacity at ambient temperature.

Generally, an actual fire consists of both heating and cooling phases. Hence, when structural elements are exposed to fire, they experience a sequence of events: the initial loading before the fire, loading during the heating phase as the fire develops, and loading during the cooling phase as the fire gradually dies out. If the elements survive the fire, their residual performance after fire exposure needs to be assessed to identify whether or not the structure is safe for ongoing service or whether any repairs are necessary to the fire-damaged member. Previous research results, as mentioned above, have indicated the excellent fire performance of CFSST columns. Some research has been performed on CFST and steel reinforced concrete (SRC) structures under combined multi-phase fire and loading. Yang et al. [20] developed a fibre model program to consider the
influence of the cooling phase on the post-fire performance of CFST columns and a formula was proposed to predict the residual strength of CFST columns after exposure to the ISO-834 standard fire including both the heating and cooling phases under an initial load. However, the interaction between the steel tube and in-filled concrete has not been analysed and full composite action was assumed by the fibre model. Song et al. [21] developed an FEA model to predict the load versus deformation relationships of CFST stub columns subjected to a combination of temperature and axial compression. The results showed that the residual post-fire ultimate strength with initial loading was slightly lower than that without initial loading, but that the peak strain corresponding to the residual ultimate strength increased significantly. Yao and Hu [22] proposed an FEA approach to evaluate the residual strength of CFST columns after natural fire exposure, and parametric analyses were also conducted. The FEA models were then extended to investigate the influence of loading and temperature histories on the post-fire behaviour of SRC column-to-SRC beam joints and CFST columns to axially and rotationally restrained steel beam joints [23-26]. Han et al. [27] developed an FEA model to study the post-fire performance of SRC columns subjected to an entire loading and fire history. User-defined subroutines were adopted to automatically choose the correct constitutive models for use during the four phases and include the influence of fire-induced explosive concrete spalling. Comparisons between predicted and test results indicated that the accuracy of the FEA models was acceptable. Limited research has however been published on the post-fire performance of CFSST columns to date. To fill this important research gap, the post-fire performance of CFSST columns is studied in the present paper by means of FEA modelling.

In this paper, an entire time (t)–load (N)–temperature (T) path, which has been adopted in previous research [20, 21, 26], is employed to investigate the influence of loading and temperature histories on the post-fire behaviour of CFSST columns. As shown in Fig. 1, the entire t–N–T history (AA'B'C'D'E') includes four phases: (1) the ambient temperature phase (AA'), in which external load (Ñ) is applied to the structural member before fire exposure; (2) the heating phase (A'B'), where the temperature of the fire increases from ambient temperature $T_o$ to a peak value $T_h$ at time $t_h$.
under constant load; (3) the cooling phase (B'C'D'), where the temperature of the fire decays to
ambient temperature at time $t_p$ under constant load, while the temperature of the member drops to
ambient temperature at time $t_d$; (4) the post-fire phase (D'E'), in which external load is increased
from $N_f$ to the failure load $N_{ur}$ while the temperature remains at ambient temperature. On the basis
of this specified $t-N-T$ path, the following research components are presented in this paper:

1. FEA models are developed to investigate the performance of CFSSST columns under the entire
   $t-N-T$ history.
2. Results from a series of fire and post-fire tests on CFSSST columns from the published literature
   are used to systematically validate the FEA models.
3. The validated FEA models are used to investigate the thermal and structural behaviour of
   CFSSST columns, the performance of which is compared to CFST columns with the equivalent
   steel ratio.
4. A parametric study is conducted using the validated FEA models to investigate the influence of
   key factors on the residual strength index of CFSSST columns after fire.

2. Finite element analysis modelling

The finite element analysis program ABAQUS [28] was adopted to simulate the behaviour of
CFSSST columns throughout an entire fire-load history. First, the temperature field model was
established to simulate the temperature development in the CFSSST columns during the heating and
cooling phases of the fire; then the temperature data were imported into the structural analysis
model to perform the post-fire analysis. The details of the temperature field and structural analysis
models are introduced below.

2.1. Temperature field analysis

The thermal properties of the steel and concrete, including thermal conductivity and specific heat,
are key to the precise prediction of the temperature field. The interfacial contact conductance
between the steel tube and concrete, convective heat transfer coefficient, emissivity of the surface
exposed to fire and the effect of water evaporation are also important.
Tao and Ghannam [15] and Han et al. [14] developed FEA models to predict the temperature field in CFSST columns, and comparisons were conducted to validate the models against relevant temperature data recorded in tests. However, the FEA models only focussed on the temperature development during the heating phase of the fire. Song et al. [21, 23] devised FEA models to simulate the temperature development in CFST columns and SRC beam-column joints in fire including both the heating and cooling phases. In the developed FEA models [21, 23], the thermal properties of the steel and concrete in the cooling phase were taken to be the same as those in the heating phase. The same approach was adopted herein with the thermal properties of stainless steel, carbon steel and reinforcement during the cooling phase taken to be the same as those in the heating phase employed by Song et al. [21, 23] and Guo [29], i.e. the thermal properties of stainless steel and carbon steel were taken from EN 1993-1-2 [30], and those of reinforcement were taken from EN 1992-1-2 [31]. For the convective heat transfer coefficient and emissivity of the surfaces exposed to fire, 35 Wm$^{-2}$K$^{-1}$ and 0.2 for stainless steel proposed by Gardner and Ng [32], and 25 Wm$^{-2}$K$^{-1}$ and 0.7 for carbon steel as given in EN 1993-1-2 [30], were used, respectively. Since the outer tube and the infilled concrete are not in perfect contact, voids infilled with water or steam exist, and contact resistance rises [33]. Ghojel [33], Tao and Ghannam [15] and Han et al. [14] have quantitatively described the contact conductance of the tube–concrete interface for CFST and CFSST columns through comparisons between the results of tests and FEA modelling. The contact conductance of the tube–concrete interface for CFSST columns reported by Han et al. [14] and CFST columns reported by Ghojel [33] was adopted in the present paper.

The thermal transfer behaviour within concrete is more complicated than within steel because of the presence of moisture. The water in the pores of the concrete can enhance the conductivity of concrete, but when the temperature reaches 100 °C, water evaporation absorbs significant amounts of heat and slows down the rate of heat transfer. Furthermore, an increase in temperature from one face of a cross-section can cause the migration of moisture towards the colder face, inducing a dynamic change in the thermal properties of the concrete. According to previous research results on
the effect of moisture on the thermal conductivity of concrete [34], it was shown that the thermal conductivity of concrete in the heating phase is larger than that in the cooling phase, since in the cooling phase the concrete is dry or has less absorbed water. Therefore, two sets of thermal conductivity and specific heat values for the concrete were assumed — one for the heating phase and another for the cooling phase. For the thermal conductivity of the concrete during the heating and cooling phases, the models proposed by Guo [29] were used. For the specific heat of the concrete during heating phase, the model in EN 1992-1-2 [31] was used, and a simple and practical method to consider the effect of water evaporation by increasing the specific heat of the concrete between 100 °C and 200 °C according to the moisture content was adopted. For the specific heat of the concrete during the cooling phase, since the concrete is dry or has less absorbed water, the specific heat of the concrete between 100 and 200°C was assumed to be the same as that at 100°C.

The thermal conductivity and specific heat of the concrete during the heating and cooling phases are shown in Fig. 2.

Since different thermal properties of concrete during the heating and cooling phases were used in the finite element analysis process, subroutines were developed to automatically choose the corresponding thermal models during the respective phases. The subroutine flow chart is shown in Fig. 3. First, a field variable in the subroutine is defined to record the current temperature phase number, with Field (1)=1 and Field (1)=2 representing the heating and cooling phases, respectively. The sign of the temperature increment $\Delta T$, i.e. the change in temperature within one increment, is used to identify the current phase, with “$\Delta T \geq 0$” indicating that the material is in the heating phase, and the material properties corresponding to “Field (1)=1” are adopted, and “$\Delta T < 0$” indicating that the material is in the cooling phase, and the material properties corresponding to “Field (1)=2” are adopted. Through this process, the subroutines could automatically select and apply the appropriate thermal properties. For the FEA model validation, the heating and cooling curves are specified to follow the measured fire curves or, if not reported, the ISO 834 standard fire curve (1999) [35]; the ISO 834 standard fire curve (1999) [35] was used throughout the parametric study.


2.2. Structural analysis

2.2.1 Material properties in different phases

The materials in the modelled structural elements can experience four different temperature phases during the entire $t-N-T$ history — ambient temperature, heating, cooling and post-fire. According to previous studies [20, 21, 27], different constitutive models for the steel and concrete should be adopted during the four phases due to the influence of temperature on the material properties.

Initially, the ambient temperature material properties apply; during the heating phase, the strength of the steel and concrete reduce as the temperature rises; in the cooling and post-fire phases, the strength of the steel recovers to some extent as the temperature reduces, and the material properties of the steel in this phase are affected by the historical maximum temperature and the current temperature, while the strength of the concrete does not recover after having been exposed to a given maximum temperature and its properties are only dependent on the peak temperature experienced [20, 21]. Elastic-plastic stress-strain properties were defined in ABAQUS for the stainless steel, carbon steel and rebars, while the concrete damaged plasticity was adopted for the in-filled concrete. Details of the material models at the different phases are introduced below.

(1) Ambient temperature and heating phases

The material properties at ambient temperature represent a special case of those during the heating phase with the temperature $T=20{}^\circ C$. The material properties of the stainless steel and carbon steel during the ambient temperature and heating phases were defined according to EN 1993-1-2 [30], while the material properties of the reinforcing bars were defined according to EN 1992-1-2 [31]. For the confined concrete, the material models reported by Song et al. [21] were employed for the ambient temperature and heating phases.

(2) Post-fire phase

Tao et al. [36] examined existing test data on carbon steel and reinforcing bars after heating and cooling to room temperature, and developed simplified stress-strain models to describe the post-fire constitutive responses; these models [36] were adopted herein.
Wang et al. [37] tested austenitic stainless steel coupons after fire exposure, and the post-fire stress-strain curves corresponding to different peak temperatures (200–1000 °C) and heat soak times (0–135 mins) were measured. Based on the test results, a post-fire stress-strain model was proposed for austenitic stainless steel [37]; this model was adopted in the present paper.

Song et al. [21] proposed a post-fire stress-strain model for concrete confined by carbon steel tubes, which considered the maximum temperature that the material experienced during fire exposure. This model [21] was adopted herein to represent the post-fire stress-strain relationship of concrete confined by stainless steel tubes, but the post-fire yield strength of carbon steel used by Song et al. [21] was replaced by that of stainless steel based on the findings of Wang et al. [37].

(3) Cooling phase

The cooling phase is a transitional phase from the heating phase to the post-fire phase. The mechanical properties of the steel and concrete in the cooling phase are affected by the current temperature and the previous maximum temperature. In the present study, the stress-strain models for stainless steel, carbon steel and reinforcing bars adopted in the cooling phase were the same as those used in the heating phase, but the yield strength, yield strain, ultimate strength and ultimate strain were linearly interpolated between the values reached in the heating phase and the values for the post-fire phase, as reported by Yang et al. [20]. For the confined concrete, the stress-strain model in the cooling phase proposed by Yang et al. [20] and Song et al. [21] was adopted.

Typical stress-strain curves for carbon steel, stainless steel, reinforcing bars and confined concrete corresponding to the different temperature phases are shown in Fig. 4. In the structural analysis model, to automate the appropriate selection of material properties during the four temperature phases, a subroutine was employed, details of which can be found in [27].

2.2.2 Modelling method of steel-concrete interface

Surface-to-surface contact in ABAQUS was adopted to simulate the interaction between the stainless steel tube and the in-filled concrete. Hard contact in the normal direction and Coulomb friction in the tangential direction were defined. For the Coulomb friction model, a friction
coefficient of 0.25 was adopted. For columns with reinforcing bars, a tie constraint was assumed between the reinforcing bars and the in-filled concrete.

2.2.3 Meshing and boundary conditions

Eight-node brick elements (C3D8R) for the concrete and endplates, 4-node shell elements (S4) for the steel tube, and 2-node truss elements (T3D2) for the reinforcing bars and stirrups were adopted in the structural analysis model. Meshing and boundary conditions of the FEA model for a simply-supported and axially-loaded CFSST column are shown in Fig. 5. The bottom end of the column was restrained against all translations and rotations about the y and z axes, while the top end of the column was restrained against translations in x and y directions and rotations about the y and z axes. Axial load was applied to the top end of the column along the z direction.

2.3. Validation of FEA modelling approach

To validate the FEA modelling approach, relevant test data on CFSST columns under fire reported by Han et al. [14] and Tao et al. [16] were utilized. Han et al. [14] tested five axially-loaded specimens in fire, while Tao et al. [16] tested 6 specimens under fire and 6 specimens after fire with or without the presence of reinforcement, as shown in Table 1. The test results, including temperature-time curves, axial deformation development, failure modes, fire resistance and post-fire load-bearing capacity, are compared with the results obtained from the FEA modelling performed in the present paper.

Comparisons between typical measured and predicted temperature (T) versus time (t) curves for the CFSST columns are shown in Fig. 6. In general, there is reasonably good agreement between the FEA predictions and the test results, as also obtained in previous studies [14, 16]. The predicted temperature–time curve of point 3 in the centre of the concrete exhibit a clear plateau at 100 °C–200 °C, mirroring the observations from the corresponding fire tests and confirming that increasing the specific heat of the concrete between 100 °C and 200 °C enables the characteristic of the water in the concrete evaporating at approximately 100 °C to be captured. Additionally, the predicted temperature–time curves during the cooling phase also agree well with the measured
results with the reduced conductivity and specific heat of concrete during the cooling phase in comparison to the heating phase enabling the relatively slow cooling rate to be captured.

Typical experimentally observed and predicted column failure modes are shown in Fig. 7. The comparisons presented in Fig. 7(a) and (b) are for fire tests [14, 16] and FEA simulations, while those in Fig. 7(c) and (d) are for post-fire tests [16] and FEA simulations. In all cases, the failure modes, including both global and local deformations, are in close agreement.

The predicted axial deformation ($A$) versus time ($t$) curves are compared with the measured results in Fig. 8 for the fire resistance and post-fire tests on CFSST columns, respectively. The predicted $A$–$t$ curves generally agree well with the test results. The predicted axial deformation, however, is generally smaller than the measured deformation in the expansion phase, particularly for specimens CT05, CT08 and ST01 [16]. This may be related to a discrepancy between the real thermal expansion coefficients of the tested stainless steel and concrete materials and the values provided in EN 1993-1-2 [30] and EN 1992-1-2 [31].

Fire resistance and residual load-bearing capacity are critical factors in the performance evaluation of CFSST columns at elevated temperature and after fire exposure, respectively. The FEA predicted fire resistance and residual load-bearing capacity of the considered test specimens are compared in Fig. 9; generally, the results are in good agreement. The relative errors between the predicted and tested fire resistance are around 10%, and the predicted divided by test fire resistances have a mean ratio of 1.048 and a COV of 0.071 as shown in Fig. 9(a); However, the relative errors between the predicted and tested residual load-bearing capacity range from -14.3% to 9.6%, as shown in Fig. 9(b).

From the above comparisons, it can be concluded that, in general, the accuracy of the adopted FEA modelling approach is acceptable. The numerical models can therefore be extended to conduct full-range analyses of CFSST columns under fire conditions and after fire exposure.

### 3. Behaviour of CFSST columns after fire

In this section, the behaviour of a typical CFSST column is examined to explore the general
response of this cross-section type in fire in comparison to a CFST column with the same total cross-sectional area of steel. The examined CFSST column has a circular cross-section of diameter 800 mm and thickness 16 mm with a height \( H \) of 6400 mm mirroring the dimensions of a real CFST column employed in a project in China [38]. The cube strength and elastic modulus of the in-filled concrete are 60 MPa and \( 3.6 \times 10^4 \) N/mm\(^2\), respectively. The stainless steel tube is formed from Grade S30408 austenitic stainless steel, which is specified in CECS 410: 2015 [39] to have a 0.2\% proof strength \( (\sigma_{0.2}) \) and ultimate strength \( (\sigma_u) \) of 205 MPa and 515 MPa, respectively; this grade corresponds approximately to Grade 1.4301 austenitic stainless steel in EN 1993-1-4: 2006+A1:2015 [40]. The yield strength and elastic modulus of the outer carbon steel tube of the comparative CFST sections, are 345 MPa and \( 2.05 \times 10^5 \) N/mm\(^2\), respectively.

For comparison purposes, pinned-pinned boundary conditions at the two ends are adopted. According to the established FEA models in Section 2, the axial ultimate load-carrying capacity at ambient temperature \( (N_u) \) and the fire resistance \( (t_R) \) of the examined columns are \( 3.43 \times 10^4 \) kN and 65 mins for the CFSST column, and \( 4.14 \times 10^4 \) kN and 42 mins for the CFST column, respectively.

To allow a direct comparison of the influence of fire on the two composite columns, both were exposed to a heating time \( t_h = 32.5 \) mins, which was determined by assessing a heating time ratio \( (t_0) \), defined as the ratio of heating time \( (t_h) \) to fire resistance of CFSST columns \( (t_R) \), of 0.5. This heating time is less than the fire resistance of both the CFSST and CFST columns, which ensures that failure would not occur during the heating or cooling phases. The applied load ratio on the two composite columns is 0.4, calculated as \( n = N_F/N_u \), where \( N_F \) and \( N_u \) are the axial load applied to the columns and the ultimate load-bearing capacity of the composite columns at ambient temperature, respectively. The ultimate load-bearing capacity of the composite columns is obtained from the FEA models. The CFSST and CFST columns are then loaded following the entire \( t-N-T \) path as shown in Fig. 1.
3.1. Temperature development in heating and cooling phases

The predicted temperatures ($T$) as a function of fire exposure time ($t$) for the CFSST and CFST columns are shown in Fig. 10, where Point 1 is located at the inner surface of the steel tube and Points 2, 3 and 4 are located at the outer surface, the mid-point of the radius and the centre point of the in-filled concrete, respectively. In the heating phase, it can be seen that the temperature of Point 1 at the inner surface of the steel tube is higher than that at Point 2 at the outer surface of the in-filled concrete, with a maximum difference of up to 243 °C for the CFSST column and 179 °C for the CFST column, and that the temperature at Point 1 of the stainless steel tube is higher than that of the carbon steel tube. This is because the heat conductance of the interface is included in the FEA models and the heat conductance between carbon steel and concrete is higher than that between stainless steel and concrete [33, 14]. Also, the thermal conductivity of stainless steel is lower than that of carbon steel below 800 °C; more heat input is therefore needed to heat the stainless steel tube under the same conditions.

Points 1-4 reach their peak temperatures during the cooling phase. It can be seen that the further the location of the point to the fire-exposed surface, the more obvious the retardation of the peak temperature. For example, as shown in Fig. 10(a), the temperatures at Points 1 and 3 reach their peak values at about 46 mins and 314 mins, while the air temperature reaches its peak temperature at 32.5 mins. It is also shown that when the temperature of the outer part of the column is dropping, the temperature of the inner part of the column can still be increasing.

3.2. Axial displacement versus time relationships in the four characteristic phases

The axial deformation ($\Delta$) versus time ($t$) curves of the CFSST and CFST columns over the full four characteristic loading–fire phases are shown in Fig. 11. The predicted axial displacement ($\Delta$) versus time ($t$) curves shown in Fig. 11 generally consist of four phases of behaviour, corresponding to the four loading–fire phases, i.e.

1) Ambient temperature loading phase (o-a). During this phase, the initial load is applied to the CFSST and CFST columns before exposure to fire, and the corresponding $\Delta$-$t$ relationship is
2) Expansive phase (a-b). During this phase, the load on the column remains constant while the room temperature increases following the ISO 834 standard fire curve [35]. As the material temperature rises, thermal expansion and degradation of strength and stiffness ensue. However, in this phase, the effect of axial thermal expansion generally exceeds the effect of material degradation, and overall expansive deformation of the column is observed. When the effects of material expansion and degradation are balanced, the expansion displacement reaches a peak value at Point b.

From Fig. 11, it can be observed that the magnitude of the expansive deformation of the CFSST column is less than that of the CFST column. The above difference can be explained by the different properties of stainless steel and carbon steel and the proportions of the applied load carried by the outer tube of the composite columns through the following points: (a) the temperature of the stainless steel tube for the CFSST column is greater than that of the carbon steel tube for the CFST column under the same fire conditions, as shown in Fig. 10; (b) the yield strength of stainless steel degrades more rapidly than that of carbon steel below 600 °C [30]; (c) the proportion of the applied load carried by the outer stainless steel tube of the CFSST column increases from 34.2% to 42.0% during this phase, while this proportion increases from 36.9% to 73.7% for the outer carbon steel tube of the CFST column, as shown in Fig. 12.

3) Softening phase (b-b'-c). When the environmental temperature reaches its maximum value and then undergoes cooling, the temperature of the steel tube and the outer part of the in-filled concrete decreases, while the temperature of the inner part of the in-filled concrete is still increasing, as shown in Fig. 10, i.e., the material performance of the steel tube and the outer part of the in-filled concrete recovers during the cooling phase while the material performance of the inner part of the in-filled concrete continues to degrade. Therefore, in the softening phase, the contractive deformation induced by the material degradation becomes dominant and the axial deformation of the column changes from expansive to contractive. After about 200 mins (b'), the
temperature of all material remains relatively constant below 100 °C. This is reflected in the $\Delta - t$ curves shown in Fig. 11, which remain approximately flat in the softening phase from 200 mins to 900 mins since the mechanical performance of the stainless steel, carbon steel and concrete is essentially constant below 100 °C.

The magnitude of the axial deformation in this phase is however greater for the CFSST column than that for the CFST column. This can be explained by the following points: (a) the historical maximum temperature of the stainless steel tube of the CFSST column is higher than that of the carbon steel tube of the CFST column under the same fire conditions, as shown in Fig. 10, and (b) the yield strength of the stainless steel initially (below 600 °C) degrades more rapidly at the elevated temperature and recovers more slowly in the cooling phase compared with that of carbon steel [30].

4) Accelerated failure phase (c-d). When the cooling phase ends, assuming the column survives the fire, the post-fire loading phase starts. In this phase, the axial load is increased until the column fails to support the axial load.

3.3. Internal forces redistribution in the entire loading and fire phase

For the axially loaded CFSST and CFST columns, the external load is supported by the steel tube and in-filled concrete together. The loads borne by the two parts are constant at ambient temperature, but, under fire conditions, the internal forces redistribute due to the nonhomogeneous degradation of material properties. To investigate the internal force redistribution in the CFSST and CFST columns over the loading–fire history, the proportions of load supported by the steel tube ($p_t$) and concrete ($p_c$) at the mid-height cross-section are extracted from the predicted results, and the proportion versus time curves are given in Fig. 12, in which Points A, A', B', C', D' and E' correspond to the feature points defined in Fig. 1. Positive or negative values of the proportion represent the specified part being in compression or tension, respectively.
Both of the two parts (i.e., the outer tube and the concrete) of the CFSST and CFST columns are in compression during the ambient temperature phase; at the end of the ambient temperature phase (Point A'), $p_t=34.2\%$ and $p_c=65.8\%$ for the CFSST column, and $p_t=36.9\%$ and $p_c=63.1\%$ for the CFST column. As the fire temperature increases, the temperature of the outer tube rises rapidly and the outer tube expands substantially more than the concrete. Therefore, the proportion of the load supported by the steel tube also increases, reaching a maximum value of $p_t=42.0\%$ for the CFSST column and $p_t=73.7\%$ for the CFST column. After reaching the peak value of $p_t$, the proportion of load borne by the outer tube subsequently begins to decrease with increasing temperature. When the environmental temperature is at its maximum value (Point B'), $p_t$ decreases to 36.3\% and 53.4\% for the CFSST and CFST columns, respectively. The cooling phase then begins, and as the temperature of the outer tube decreases, the steel contracts. The proportion of load carried by the outer tube therefore decreases from positive values (i.e., compressive stress) to negative values (i.e., tensile stress). From 200 mins to 900 mins, the temperature of the materials remains below 100°C as described in Section 3.2, and the predicted proportion versus time curves stay approximately constant with $p_c=162.2\%$ and $p_t=-62.2\%$ for the CFSST column, and $p_c=141.3\%$ and $p_t=-41.3\%$ for the CFST column.

When the temperature of all the materials in the CFSST and CFST columns drops to ambient temperature (Point D'), the material properties recover to some extent. The axial load is then increased until the failure of the CFSST and CFST columns by global buckling (Point E'), while the proportion of load carried by the outer tube changes from -62.2\% to 9.1\% for the CFSST column and from -41.3\% to 11.0\% for the CFST column.

### 3.4. Failure modes

The studied CFSST and CFST columns have a slenderness ratio $\lambda$ of 32 where $\lambda=4 H/D$, in which $H$ is the effective bucking length of column and $D$ is the diameter of the cross-section. The failure modes under the post-fire loading, as shown in Fig. 13, show global buckling, accompanied by
some local buckling of the steel tube near the mid-height of the column. Local buckling failure is however possible during different phases of the fire owing to the relative lateral displacement between the steel tube and the in-filled concrete, the varying proportions of load borne by the steel tube and the varying levels of material degradation.

In order to explore the above phenomenon, the mid-height cross-section is isolated, and the relative lateral displacement between Point 1 at the inner surface of the stainless tube and Point 2 at the outer surface of concrete shown in Fig. 13 is extracted. The relative lateral displacement between Points 1 and 2 firstly increases in the heating phase. The maximum value reaches 9.5 mm for the CFSST columns and 6.5 mm for the CFST columns. The steel tube contracts with the decreasing temperature of the steel tube during the cooling phase, and the relative lateral displacement reduces to 0.1 mm by the end of the cooling phase for both the CFSST and CFST columns. In the post-fire phase, the relative lateral displacements at failure are 1.2 mm and 0.8 mm for the CFSST and CFST columns, respectively.

3.5. Post-fire residual strength index

The predicted axial load ($N$) versus axial deformation ($\Delta$) curves of the CFSST and CFST columns are shown in Fig. 14, in which Points o, a, b, c and d correspond to those points in Fig. 11, and $d_p$ is the peak point of the $N-\Delta$ curves after fire. Moreover, for the comparison purposes, the axial load ($N$) versus axial deformation ($\Delta$) curves of the CFSST and CFST columns at ambient temperature are also presented in Fig. 14, in which, $d'_p$ is the peak point of the $N-\Delta$ curves at ambient temperature and $d'$ is the point of failure of the columns, which is deemed to occur when the limiting axial contraction and the limiting rate of axial contraction specified in ISO 834-1 [35] are exceeded. It can be seen from Fig. 14 that the peaks of the $N-\Delta$ curves of the CFSST and CFST columns after fire are lower than those at ambient temperature and that the axial deformation corresponding to the peak axial load are larger than those at ambient temperature; this shows the load-bearing capacity and corresponding deformation of the CFSST and CFST columns under combined fire and loading are reduced and increased, respectively, to some degree, relative to ambient temperature loading.
alone.

In order to evaluate the residual load-bearing capacity of the CFSST and CFST columns, a residual strength index $k_r$ is defined, which is expressed as follows:

$$k_r = \frac{N_{ur}(t_h)}{N_u}$$  \hspace{1cm} (1)

where $N_u$ is the ultimate axial capacity at ambient temperature, as indicated by Point $d'_p$ in Fig. 14 and $N_{ur}(t_h)$ is the post-fire residual load-bearing capacity after heating time $t_h$, as indicated by Point $d_p$ in Fig. 14. Both of the two parameters are determined from the FEA models.

It can be seen from Fig. 14 that the residual strength indices $k_r$ for the studied CFSST and CFST columns are 0.946 and 0.865, respectively, and the corresponding axial deformations are 52.4 mm and 46.1 mm, respectively. It can be concluded that the residual load-bearing capacity of the studied CFSST column is superior to that of the CFST column under the same fire conditions.

4. Parametric analysis into the residual strength index

The residual strength index shown in Eq. (1) is a critical parameter for evaluating the post-fire residual load-bearing capacity of CFSST columns. Based on the FEA models developed in Section 2, parametric analysis to investigate the residual strength index of CFSST columns is conducted in the present section. The influence of three types of parameter — geometrical, material and loading are investigated, and the critical parameters are identified.

4.1. Influencing parameters and basic models

A comprehensive parametric study is performed to investigate the influence of different parameters on the residual strength index $k_r$. The potential influencing parameters are listed in Table 2 and detailed below:

(1) Geometric parameters: diameter of the stainless steel tube $D$; column slenderness $\lambda$, where $\lambda = \frac{H}{D}$, with being $H$ is the effective bucking length of the column and steel ratio $\alpha$, where $\alpha = \frac{A_s}{A_c}$, and $A_s$ and $A_c$ are the cross-sectional areas of the stainless steel tube and concrete,
respectively.

(2) Loading parameters: heating time ratio $t_0$, load eccentricity ratio $e/r$, in which $e$ is load eccentricity and $r=D/2$ and load ratio $n$, where $n=N_e/N_u$.

(3) Material parameters: yield strength of the stainless steel tube $\sigma_{0.2}$, though retaining the physical and thermal properties of austenitic stainless steel throughout the study and concrete cube strength $f_{cu}$.

The full $t$–$N$–$T$ history is used in the parametric analysis, in which the heating time ratio $t_0$ varies from 0 to 0.6 at intervals of 0.1. The parameters in bold font in Table 2 are those used in the reference models of the CFSST column described in Section 3.

4.2. Influence of key parameters

The residual strength indices $k_r$ corresponding to variation in the key parameters are shown in Fig. 15. It can be seen from Fig. 15 that $k_r$ generally decreases with increasing heating ratios, but the range in $k_r$ values (i.e. the difference between the maximum and minimum $k_r$ values for a given value of $t_0$) varies, depending on the parameter under consideration. The varied parameters are considered in three categories based on the magnitude of their influence on $k_r$.

The first category includes load ratio $n$, diameter of stainless steel tube $D$ and slenderness $\lambda$. As shown in Fig. 15(a), $k_r$ decreases with increasing $n$. This is because larger $n$ values result in higher loads in both the stainless steel tube and the in-filled concrete, inducing more severe deformation during the fire, thus harming the post-fire performance. The influence of $n$ on $k_r$ for CFSST columns is similar to that observed for CFST columns by Yang et al. [20]. As shown in Fig. 15(b), $k_r$ increases with increasing values of $D$. This is mainly because a larger diameter means a larger cross-section of concrete and therefore a higher specific heat volume. As shown in Fig. 15(c), $k_r$ decreases with increasing slenderness $\lambda$, because CFSST columns with larger slendernesses are more sensitive to second order effects and have higher residual lateral deflections, which damage the post-fire performance and reduce $k_r$. In this category, the deviation between the minimum and maximum $k_r$ values corresponding to variation in the three parameters is more than 10%, i.e. these
three parameters have a major influence on $k_r$.

The second category includes steel ratio $\alpha$ and load eccentricity ratio $e/r$. As shown in Fig. 15(d), $k_r$ increases with increasing values of $\alpha$, though is approximately constant for $\alpha=8.5\%$ and above. This is because the confinement effect afforded by the steel tube to the in-filled concrete increases with increasing $\alpha$ values. As shown in Fig. 15(e), $k_r$ decreases with increasing $e/r$ values, because the residual lateral deflections are enhanced and hence the post-fire performance is degraded. The difference in post-fire performance due to variation in the steel ratio $\alpha$ and the eccentricity ratio $e/r$ is between 5\% and 10\%, i.e. these two parameters have a moderate influence on $k_r$.

The third category includes concrete cube strength $f_{cu}$ and yield strength of the stainless steel tube $\sigma_{0.2}$, as shown in Figs. 15(f) and (g). In this category, the difference between the minimum and maximum $k_r$ values corresponding to variation in $f_{cu}$ and $\sigma_{0.2}$ is below 5\%, i.e. these two parameters have only a minor influence on $k_r$.

From the above parametric analysis and discussion, it can be concluded that load ratio $n$, diameter of stainless steel tube $D$ and column slenderness $\lambda$ significantly influence the residual strength index $k_r$ and should be included in design proposals for the practical calculation of post-fire resistance, while the results indicate that the influence of the other parameters could be ignored for simplicity.

On this basis, a design table to determine the post-fire residual strength index $k_r$ of circular CFSST columns using austenitic stainless steel as a function of the key identified parameters is proposed, as shown in Table 3. The validity limits of application for Table 3 are: $D=200–2000$ mm, $\lambda=8–120$, $n=0.2–0.8$, $t_o=0–0.6$, $f_{cu}=40–80$ MPa, $\sigma_{0.2}=205–400$ MPa (austenitic stainless steel), $\alpha=4\%–15\%$ and $e/r=0–1.2$.

5. Conclusions

The following conclusions can be drawn from the research described in the present paper:

(1) An FEA model was established to simulate the behaviour of CFSST columns subjected to a full loading–fire history. To improve the accuracy of the FEA modelling, the influence of moisture
on the thermal conductivity and specific heat of concrete during the heating and cooling phases was considered through user subroutines. Existing fire resistance and post-fire test data on CFSST columns were used to validate the FEA modelling approach. Comparisons between the FEA predicted and test results indicated that the accuracy of the FEA model is acceptable.

(2) The residual strength index of CFSST columns was found to be higher than that of CFST columns after exposure to the same fire conditions despite the axial deformation and the historical maximum temperature being higher.

(3) Parametric analyses into the influence of a series of parameters on the post-fire residual strength index of CFSST columns were conducted. Four parameters — diameter of the cross-section, member slenderness, heating time ratio and load ratio — were identified as having a significant influence on the residual strength index.

Acknowledgements

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References


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Captions for Tables

**Table 1** Summary of fire tests of the CFFST columns

**Table 2** Parameters varied in parametric analysis.

**Table 3** Design table for residual strength index $k_r$ for circular CFSST columns using austenitic stainless steel.
<table>
<thead>
<tr>
<th>Types of columns</th>
<th>Types of section</th>
<th>Number of specimens</th>
<th>Sectional dimension $B \times t$ (mm × mm)</th>
<th>Height (mm)</th>
<th>$f'_c$ (MPa)</th>
<th>Slenderness</th>
<th>Load ratio</th>
<th>Types of tests</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFSST columns</td>
<td>Circular</td>
<td>2</td>
<td>300×5</td>
<td>3700</td>
<td>54.8</td>
<td>49.3</td>
<td>0.30, 0.45</td>
<td>Fire resistance</td>
<td>Han et al. (2013) [14]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>200×3</td>
<td>1870</td>
<td>41.4, 42.3</td>
<td>37.4</td>
<td>0.31, 0.48</td>
<td>Fire resistance</td>
<td>Tao et al. (2016) [16]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>200×3</td>
<td>1870</td>
<td>44.3, 44.8</td>
<td>37.4</td>
<td>0.32, 0.48</td>
<td>Post-fire</td>
<td>Tao et al. (2016) [16]</td>
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<tr>
<td></td>
<td>Circular</td>
<td>2</td>
<td>315×5</td>
<td>3700</td>
<td>54.8</td>
<td>40.7</td>
<td>0.15, 0.30</td>
<td>Fire resistance</td>
<td>Han et al. (2013) [14]</td>
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<td>Square</td>
<td>1</td>
<td>630×10</td>
<td>3700</td>
<td>54.8</td>
<td>20.3</td>
<td>0.30</td>
<td>Fire resistance</td>
<td>Han et al. (2013) [14]</td>
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<td></td>
<td></td>
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<td>200×4</td>
<td>1870</td>
<td>43.5, 44.1</td>
<td>32.4</td>
<td>0.31, 0.46</td>
<td>Fire resistance</td>
<td>Tao et al. (2016) [16]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>200×4</td>
<td>1870</td>
<td>46.4, 46.6</td>
<td>32.4</td>
<td>0.31, 0.45</td>
<td>Post-fire</td>
<td>Tao et al. (2016) [16]</td>
</tr>
<tr>
<td>CFSST columns with reinforcements</td>
<td>Circular</td>
<td>2</td>
<td>200×3</td>
<td>1870</td>
<td>41.7, 42.3</td>
<td>37.4</td>
<td>0.28, 0.42</td>
<td>Fire resistance</td>
<td>Tao et al. (2016) [16]</td>
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<tr>
<td></td>
<td></td>
<td>2</td>
<td>200×3</td>
<td>1870</td>
<td>44.8, 45.5</td>
<td>37.4</td>
<td>0.28, 0.43</td>
<td>Post-fire</td>
<td>Tao et al. (2016) [16]</td>
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Table 2 Parameters varied in parametric analysis.

<table>
<thead>
<tr>
<th>Type of parameters</th>
<th>Name of parameters</th>
<th>Symbols</th>
<th>Units</th>
<th>Values</th>
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<tr>
<td>Geometric parameters</td>
<td>Diameter of stainless steel tube</td>
<td>$D$</td>
<td>mm</td>
<td>200, 600, <strong>800</strong>, 1200, 1600, 2000</td>
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<tr>
<td></td>
<td>Slenderness</td>
<td>$\lambda$</td>
<td>-</td>
<td>8, 16, <strong>32</strong>, 64, 96, 120</td>
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<tr>
<td></td>
<td>Steel ratio</td>
<td>$\alpha$</td>
<td>-</td>
<td>4%, 6%, <strong>8.5%</strong>, 10%, 12%, 15%</td>
</tr>
<tr>
<td>Loading parameters</td>
<td>Heating time ratio</td>
<td>$t_0$</td>
<td>-</td>
<td>0, 0.1, 0.2, 0.3, 0.4, <strong>0.5</strong>, 0.6</td>
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<tr>
<td></td>
<td>Load eccentricity ratio</td>
<td>$e/r$</td>
<td>-</td>
<td>0, 0.3, 0.6, 0.9, 1.2</td>
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<tr>
<td></td>
<td>Load ratio</td>
<td>$n$</td>
<td>-</td>
<td>0.2, <strong>0.4</strong>, 0.6, 0.8</td>
</tr>
<tr>
<td>Material parameters</td>
<td>Yield strength of stainless steel tube</td>
<td>$\sigma_{0.2}$</td>
<td>MPa</td>
<td><strong>205</strong>, 300, 400</td>
</tr>
<tr>
<td></td>
<td>Concrete cube strength</td>
<td>$f_{cu}$</td>
<td>MPa</td>
<td>40, <strong>60</strong>, 80</td>
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</table>
Table 3 Design table for residual strength index $k_r$ for circular CFSST columns using austenitic stainless steel.

<table>
<thead>
<tr>
<th>$D$ (mm)</th>
<th>$\lambda$</th>
<th>$n$</th>
<th>$t_o$</th>
<th>$k_r$</th>
<th>$D$ (mm)</th>
<th>$\lambda$</th>
<th>$n$</th>
<th>$t_o$</th>
<th>$k_r$</th>
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<tbody>
<tr>
<td>200</td>
<td>32</td>
<td>0.4</td>
<td>0.3</td>
<td>0.74</td>
<td>800</td>
<td>32</td>
<td>0.2</td>
<td>0.3</td>
<td>0.98</td>
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<tr>
<td>0</td>
<td>1.00</td>
<td></td>
<td></td>
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<td>0.1</td>
<td>0.88</td>
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<td>0.1</td>
<td>0.99</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>0.81</td>
<td></td>
<td></td>
<td></td>
<td>0.2</td>
<td>0.99</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| 200      | 32        | 0.4 | 0.3  | 0.83 | 800      | 32        | 0.6 | 0.3  | 0.93 |
| 0        | 1.00      |     |      |      | 0        | 1.00      |     |      |      |
| 0.1      | 0.94      |     |      |      | 0.1      | 0.99      |     |      |      |
| 0.2      | 0.88      |     |      |      | 0.2      | 0.98      |     |      |      |

| 800      | 32        | 0.4 | 0.3  | 0.98 | 800      | 32        | 0.8 | 0.3  | 0.93 |
| 0        | 1.00      |     |      |      | 0        | 1.00      |     |      |      |
| 0.1      | 0.99      |     |      |      | 0.1      | 0.99      |     |      |      |
| 0.2      | 0.98      |     |      |      | 0.2      | 0.98      |     |      |      |

| 1200     | 32        | 0.4 | 0.3  | 0.97 | 800      | 64        | 0.4 | 0.3  | 0.91 |
| 0        | 1.00      |     |      |      | 0        | 1.00      |     |      |      |
| 0.1      | 0.99      |     |      |      | 0.1      | 0.99      |     |      |      |
| 0.2      | 0.98      |     |      |      | 0.2      | 0.95      |     |      |      |
**Captions for Figures**

Fig. 1. Time ($t$)–load ($N$)–temperature ($T$) path.

Fig. 2. Thermal properties of concrete during heating and cooling phases.

Fig. 3. User subroutine flow chart for automatic selection of thermal properties in heating and cooling phases.

Fig. 4. Material stress-stain curves in different temperature phases.

Fig. 5. Meshing and boundary conditions of FEA model for a simply-supported and axially-loaded CFSST column.

Fig. 6. Typical comparison between test and FEA temperature ($T$) versus time ($t$) curves for CFSST columns.

Fig. 7. Comparison between observed and predicted column failure modes.

Fig. 8. Typical comparison between test and FEA axial deformation ($\Delta$) versus time ($t$) curves for CFSST columns.

Fig. 9. Comparison between test and predicted fire resistances and residual load-bearing capacity of the CFSST columns.

Fig. 10. Temperature ($T$) versus time ($t$) curves for CFSST and CFST columns during and after fire.

Fig. 11. Typical axial deformation ($\Delta$) versus time ($t$) curves for CFSST and CFST columns during and after fire.

Fig. 12. Proportion of load borne by each part of CFSST and CFST columns.

Fig. 13. Failure modes of CFSST and CFST columns under post-fire loading.

Fig. 14. Axial load ($N$) versus axial deformation ($\Delta$) curves for CFSST and CFST columns under post-fire loading (Comparative ambient temperature $N$–$\Delta$ curves are also shown).

Fig. 15. Influence of parameters on $k_r$ for CFSST columns under post-fire loading.
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Fig. 2. Thermal properties of concrete during heating and cooling phases.
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Fig. 8. Typical comparison between test and FEA axial deformation ($\Delta$) versus time ($t$) curves for CFSST columns.
Fig. 9. Comparison between test and predicted fire resistances and residual load-bearing capacity of CFSST columns.
Temperature decay to ambient temperature
End of heating phase
Temperature of fire decays to ambient temperature
End of cooling phase

(a) CFSST column
(b) CFST column

Fig. 10. Temperature ($T$) versus time ($t$) curves for CFSST and CFST columns during and after fire.
Fig. 11. Typical axial deformation ($\Delta$) versus time ($t$) curves for CFSST and CFST columns during and after fire.
Fig. 12. Proportion of load borne by each part of CFSST and CFST columns.
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Fig. 14. Axial load ($N$) versus axial deformation ($\Delta$) curves for CFSST and CFST columns under post-fire loading; comparative ambient temperature $N$-$\Delta$ curves are also shown.
(g) Yield strength of stainless steel tube ($\sigma_{0.2}$)

**Fig. 15.** Influence of parameters on $k_r$ for CFSST columns under post-fire loading.
Research Highlights

1. FEA model established to simulate fire and post-fire performance of CFSST columns.

2. User subroutines developed for automatic selection of thermal properties in heating and cooling phases.

3. Behaviour of CFSST column during full loading–fire history explored and compared to CFST column with the same steel ratio.

4. Parametric analysis on residual strength index of CFSST columns after fire conducted and design tables proposed.