

Post-fire behaviour of reinforced concrete stub columns confined by circular steel tubes

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Abstract:

Experimental and numerical studies into the structural behaviour of reinforced concrete columns confined by circular steel tubes after exposure to standard fire conditions are presented. These elements differ from conventional concrete-filled steel tubular (CFST) columns in that breaks in continuity are present at the member ends, which limit the longitudinal stresses in the steel and maximise the level of confinement afforded to the concrete through the generation of hoop stresses in the tube. The temperature distributions in the specimens were measured during the heating and cooling phases, while the load-displacement relationships and longitudinal and transverse strains in the steel tube were recorded during the subsequent compressive tests. 3D finite element (FE) models were also developed using the program ABAQUS to investigate the post-fire performance of circular steel tube confined reinforced concrete (CSTCRC) columns, including both heat transfer and stress analyses. The FE models were used to identify the influences of key parameters on the residual capacity of CSTCRC columns, following exposure to the ISO-834 standard fire. The considered parameters included heating time, cross-sectional dimensions, strength of the materials, steel tube to concrete area ratio and ratio of reinforcement. Finally, a design method was proposed for predicting the residual load bearing capacity and compressive stiffness of CSTCRC columns after standard fire exposure.

Keywords: circular steel tube confined reinforced concrete; compressive stiffness; post-fire; residual capacity; thermal distribution

Nomenclature

A_b	cross-sectional area of reinforcing bars
A_c	cross-sectional area of concrete core
A_s	cross-sectional area of steel tube
A	cross-sectional area of composite section, $A = A_s + A_c + A_b$
d_s	diameter of bars
D	outer diameter of the cross-section
EA	compressive stiffness of column
E_b	Young's modulus of reinforcement at ambient temperature
E_{bT}	Young's modulus of reinforcement after fire exposure
E_c	Young's modulus of concrete at ambient temperature
E_{cT}	Young's modulus of concrete after fire exposure
E_s	Young's modulus of steel at ambient temperature
E_{sT}	Young's modulus of steel after fire exposure
f_b	yield strength of reinforcement at ambient temperature
f_{bu}	ultimate tensile strength of reinforcement
f_{bT}	yield strength of reinforcement after fire exposure
f_{ck}	characteristic concrete strength, $f_{ck} = 0.67 f_{cu}$
f_{cu}	concrete cube strength
$f_{cu,28}$	concrete cube strength at 28 days
$f_{cu,test}$	concrete cube strength at the test day of the specimens
f'_c	concrete cylinder strength
f'_{cT}	concrete cylinder strength after fire exposure
f'_{tT}	concrete tensile strength after fire exposure
f_{su}	ultimate tensile strength of structural steel
f_y	yield strength of steel at ambient temperature
f_{yT}	yield strength of steel after fire exposure
k	factor accounting for the delay of temperature rise of concrete
L	length of column

N_y	yield load of composite column
N_u	ultimate capacity of composite column
t_h	heating time to the maximum fire temperature
t_s	wall thickness of the steel tube
T	temperature
$T_{\max}$	the maximum temperature achieved during the heating and cooling phases
α_b	ratio of reinforcement, $\alpha_b = A_b / (A_c + A_b)$
α_s	steel tube to concrete area ratio, $\alpha_s = A_s / A_c$
ε_{bf}	percentage elongation at fracture of reinforcement
ε_{sf}	percentage elongation at fracture of structural steel
μ	friction coefficient
μ_Δ	ductility index
ν_s	Poisson's ratio of structural steel
ξ	confinement factor, $\xi = f_y A_s / f_{ck} A_c$
σ_v	longitudinal stress in steel tube
σ_h	transverse stress in steel tube
σ_z	equivalent stress in steel tube, $\sigma_z = \frac{\sqrt{2}}{2} \sqrt{(\sigma_v - \sigma_h)^2 + \sigma_v^2 + \sigma_h^2}$
τ_{bond}	bond stress
τ_{crit}	critical shear stress
Δ_y	displacement at yield load
Δ_u	displacement at ultimate load
$\Delta_{0.85}$	displacement at $0.85N_u$ after peak load

1. Introduction

The steel tube confined reinforced concrete (STCRC) column is a composite member in which the steel tube confines the reinforced concrete, but with a key feature being the breaks in continuity of the steel tube at the member ends. A schematic view of a STCRC column and its beam to column connection is shown in Fig.1. The steel tube acts as formwork and operates as hoop reinforcement, such that the confinement to the core concrete can be maximized and the possibility of local buckling of the steel tube is reduced, compared with conventional concrete-filled steel tubular (CFST) columns. The confinement effect begins from the onset of loading, with the concrete in a triaxial compression state, and offers enhanced load bearing capacity and ductility. The steel tube is terminated at the beam to column connection in order to prevent it from directly bearing longitudinal force. The reinforced concrete beam to STCRC column connections can therefore be designed and constructed following conventional methods for reinforced concrete structures, which overcomes construction difficulties associated with connecting reinforced concrete beams to composite columns. Although similar steel ratios are needed for STCRC columns as for CFST columns, thinner steel tubes are preferred in the former case, with the steel tube to concrete area ratio (A_s/A_c) ideally ranging from about 2% to 4% and the additional steel used as main reinforcements embedded in the core concrete. This approach enhances the fire performance of STCRC columns.

The steel tube confined plain concrete column was first studied as a means of loading circular CFST columns by Gardner and Jacobson [1], Orito et al. [2], Prion and Boehme [3], O'Shea and Bridge [4] and Fam et al. [5]. The concept of the steel tube confined reinforced concrete column was first proposed by a research group led by Tomii [6,7], with the aim of preventing shear failure and improving the ductility of reinforced concrete stub columns. Experimental studies were conducted to investigate the axial compressive performance [8] and seismic performance [6,7] of this kind of member, yielding very promising results. Liu and Zhou [9,10] conducted experiments on the axial compressive behaviour and seismic behaviour of circular and square shaped STCRC columns and proposed design methods for predicting the axial load bearing capacity and shear capacity of these columns. Han et al. [11] conducted experiments on the seismic performance of reinforced concrete beam to STCRC column connections, confirming their good seismic performance and applicability in practical building structures.

Previous studies have focused on the mechanical behaviour of STCRC columns at ambient temperature; there is no reported research on this topic that is relevant to high temperatures. Hence, the behaviour of circular steel tube confined reinforced concrete (CSTCRC) stub columns after exposure to the ISO-834 standard fire [12] were studied experimentally and numerically in this paper. Temperatures of the cross-section during the heating and cooling phases, axial load versus displacement curves, longitudinal and transverse strains in the steel tube and the observed failure modes were all measured and discussed. Additionally, a finite element (FE) model was developed with the program ABAQUS, adopting a sequentially coupled thermal-stress analysis, and validated against the tests. Parametric studies were then performed to identify the influences of key parameters on the post-fire behaviour of CSTCRC columns, including exposure time, cross-sectional dimensions and strength of the materials. Based on the parametric studies, a simplified method was proposed to predict the residual capacity and compressive stiffness of CSTCRC stub columns after exposure to a standard fire.

2. Experimental study

2.1 Specimens

A total of 21 circular steel tube confined reinforced concrete (CSTCRC) stub columns were tested. The key parameters considered were heating time (the time corresponding to the maximum fire temperature being reached), cross-sectional dimension and the compressive strength of the concrete. Details of the specimens are shown in Table 1, in which D is the cross-sectional diameter, t_s is the wall thickness of the steel tube, α_s is the steel tube to concrete area ratio, L is the length of specimens, α_b is the reinforcement ratio and t_h is the heating time. An example of the test specimen identification system is as follows: C250-30-30-a, where C denotes a composite column, the next three digits refer to the steel tube outer diameter in mm, the following two digits are the nominal concrete cube strength at ambient temperature in N/mm², the next two digits refer to the heating time in minutes, and the final letter identifies different specimens in a specimen group with the same parameters. For all specimens, the steel tube to concrete area ratio ($\alpha_s = A_s/A_c$) and the ratio of reinforcement ($\alpha_b = A_b/(A_c + A_b)$) were maintained constant, with nominal values of 3.62% and 3.98%, respectively. The ratio of length to cross-sectional diameter was 3 for all specimens to ensure short column behaviour.

Two enlarged end plates with a thickness of 10 mm were welded to the top and bottom of each column. Two strips were cut from the outer steel tube, 50 mm away from both ends, introducing gaps, each with a width of 10 mm, at the ends of the members. Therefore only the concrete loaded condition can be attained. Six reinforcing bars were embedded in the concrete, with a concrete cover from the edge of the concrete to the outer perimeter of the reinforcing bars of 20 mm. Stirrups, with a diameter of 8 mm and at intervals of 200 mm, were used to erect the reinforcing bars. Details of the specimens are shown in Fig.2.

The circular steel tubes used for the construction of the CSTCRC specimens were cold-formed and seam welded. The key ambient temperature properties of the steel tube, after fire exposure times of 0, 30 and 60 minutes, were determined by performing tensile coupon tests in accordance with the Chinese Standard GB/T228-2010 [13]. The results are shown in Table 2, in which t_s is the thickness of the coupon extracted from the steel tube, t_h is the heating time, E_s is the elastic modulus, f_y is the yield strength, f_{su} is the ultimate tensile strength, ν_s is the Poisson's ratio and ε_{sf} is the percentage elongation at fracture of the steel over the standard gauge length = $5.65\sqrt{S_0}$ [13], where S_0 is the cross-sectional area of the coupon. Representative stress-strain curves for the steel after different exposure times are shown in Fig.3. Strain hardening of the steel commenced when the strains reached about 3.5% and 4.0% for the unexposed and exposed conditions respectively, neither of which were achieved during the CSTCRC specimen tests. Therefore an elastic-perfectly plastic stress-strain relationship is adopted for the steel tube in later analyses. Hot-rolled ribbed steel bars and hot-rolled plain steel bars were used for the longitudinal reinforcing bars and stirrups, respectively. The diameter d_s , elastic modulus E_b , yield strength f_b , ultimate tensile strength f_{bu} and percentage elongation at fracture ε_{bf} over the standard gauge length of the reinforcing steel are presented in Table 3.

Ready mixed aggregate concrete was used to fill the steel tubes. In order to determine the compressive strength and elastic modulus of the concrete, sets of concrete cubes and concrete prisms were prepared and cured in the same condition as the specimens. The concrete cube strength f_{cu} and elastic modulus E_c were determined by testing at 28 days and at the test day of the CSTCRC specimens (Table 4).

Three stub columns of 500 mm length were prepared to measure the temperature distributions in the

specimens during the heating and cooling phases. Two of the columns had an outer diameter of 250 mm, while the third one had an outer diameter of 200 mm. These specimens are referred to as C250-30min, C250-60min and C200-30min respectively. The steel tube, the concrete infill, the reinforcing bars and the stirrups of the three stub columns were all the same as those of the corresponding test specimens. Two pairs of holes, 30 mm in diameter, located 150 mm from each end of the columns, were drilled in the wall of the steel section for the entry of thermocouples. Type K chromel-alumel thermocouples, with a diameter of 1.0 mm, were used for measuring temperatures at different locations in the cross-section and at different heights along the column. The thermocouples in the concrete were tied to ceramic rods (about 8 mm in diameter) and fixed with high-strength no-shrinkage grout in the 30 mm holes. There were also thermocouples attached to the steel tube and the reinforcing bars to measure their temperatures. The locations of the thermocouples are shown in Fig.4.

2.2 Test conditions and procedure

The experiments were conducted in a furnace that was specially built for testing structural members at Harbin Institute of Technology, in which the combined actions of elevated temperature and structural load can be applied. It has a 4.9 m × 2.6 m floor area and a height of 4.05 m. A general view of the furnace is shown in Fig.5(a). The interior faces of the chamber were lined with ceramic fibre materials in order to transfer heat to specimens efficiently. Ten gas burners were placed spirally in the furnace chamber, each of which can be adjusted individually, to generate a uniform temperature. The pressure in the furnace chamber was controlled automatically, and was set according to the ISO-834 Standard [12] and Chinese Standard GB/T 9978-2008 [14]. Eight platinum-rhodium thermocouples were installed at various heights to measure furnace temperatures. The ambient temperature at the start of the tests was 23 °C and 20 °C respectively for the 30 minutes heating and 60 minutes heating times. The furnace temperature was increased following the ISO-834 standard fire curve, including the cooling phase [12]. The specimens were placed in the furnace in an unstressed condition, which is considered to be more conservative than under load in terms of evaluating the residual strength of concrete after fire [15-18]. The top and bottom ends of the specimens, including the stub columns for measuring temperatures, were fully protected with ceramic fibre blankets to prevent heat being transferred into the specimens through the end plates.

The temperatures of the furnace, steel tube, concrete and reinforcing bars were measured during the heating and cooling phases. A high temperature camera monitored the specimens throughout the test process. After fire exposure, the tests were paused while the specimens cooled to approximately ambient temperature.

Concentric compressive loading was then applied to the specimens using a 10000 kN hydraulic compression machine (Fig.5(b)). The loading rate was 2 kN/s up to approximately 85% of the expected load bearing capacity, after which the rate was controlled by a strain rate of 600 $\mu\epsilon/\text{min}$. Linear variable displacement transducers (LVDTs), located at the top and bottom end plates, were used to measure the overall axial displacement. These LVDTs were also monitored at the early stages of loading to ensure that uniform compressive loading was applied to the columns. Pairs of strain gauges arranged at 90° were placed on the outer surface of the steel tube at the mid-length and at the edge of the top gap to monitor the longitudinal and transverse strains. The layouts of these instruments are shown in Fig.2. During the test, the axial load, axial displacement and longitudinal and transverse strains in the steel tube were monitored and recorded.

2.3 Test results and discussions

Fig.6 depicts typical failure modes of the tested circular steel tube confined reinforced concrete columns. All the specimens exhibited shear failures. It also can be observed that the concrete becomes more brittle and suffers more serious damage with increased exposure times.

The measured temperature-time curves of the furnace, which closely follow the ISO-834 standard temperature-time curve [12], are shown in Fig.7. The temperatures of the specimens during the heating and cooling phases are presented in Fig.8. Temperatures are missing for some locations because the corresponding thermocouples failed during the tests, including thermocouple 12 on specimen C250-60min and thermocouples 2, 3, 4, 8 and 9 on specimen C200-30min. The temperatures of the specimens clearly depend on the furnace temperature, which initially rises and then drops, but it can also be seen in Fig.8 that, as expected, the deeper the location of the thermocouples, the lower the maximum temperature achieved, and the longer it takes to reach this maximum temperature. For specimen C250-30min, the maximum temperatures of the steel tube, reinforcing bar and concrete centre are 643 °C, 371 °C and 238 °C, respectively, achieved at 34, 58 and 155 minutes, respectively. The delay of temperature rise in the concrete can be attributed to the

high thermal capacity of concrete and to the protection afforded to it by the surrounding material; it can be expected that this phenomenon would be more prominent for larger scale members. Temperature uniformity along the longitudinal direction of the columns was confirmed by observing the corresponding temperature data at different heights (Fig.8). Similarly, the symmetry of the temperatures at the reference cross-sections confirms that a logical thermal distribution was achieved and recorded during the heating and cooling stages.

The axial load versus axial displacement responses of the specimens are illustrated in Fig.9. In general, the initial behaviour was linear elastic up to approximately 0.9 times the peak load, followed by a decreased stiffness to the peak load, after which, the load dropped gradually. For all test specimens, Table 5 shows the compressive stiffness (EA), the yield load (N_y), the displacement at the yield load (Δ_y), the peak load (N_u), the displacement at the peak load (Δ_u), the displacement at $0.85N_u$ after the peak load ($\Delta_{0.85}$) and the ductility index (μ_Δ), which is a measure of deformation capacity defined by Eq.(1). These loads and displacements were determined as shown in Fig.10.

$$\mu_\Delta = \frac{\Delta_{0.85}}{\Delta_u} \quad (1)$$

The influences of exposure time and concrete strength on the average load bearing capacity, compressive stiffness and ductility index of the C250 test specimens are shown in Fig.11. The residual capacity (N_u) and the compressive stiffness (EA) decrease with increasing exposure times, of which the compressive stiffness is more sensitive to the exposure. Considering the specimens in group C250-50 for example, the load bearing capacity decreases by 5.7% and 15.6% for specimens exposed to fire for 30 minutes and 60 minutes respectively relative to the unexposed specimens, while the corresponding compressive stiffness decreases by 29.0% and 65.7%, respectively. This coincides with the influence of fire on the mechanical properties of the concrete, whereby the elastic modulus reduces more significantly than the strength after exposure. As expected, higher load bearing capacities and lower ductility indices are obtained with the higher strength concrete. The ductility index (μ_Δ) generally tends to decrease with increased exposure times, though it shows a small increase for 30 minutes of exposure, compared to the unexposed case (Fig.11(c)).

In order to analyse the development of stresses in the steel tube, the longitudinal and transverse strains in the steel tube were measured during the loading process. The elastic-plastic analysis method described in [19] was employed to calculate the stresses in the steel tube from the measured

strains. The elastic-perfectly plastic stress-strain relationship was assumed for the steel tube. Fig.12 shows typical analysis results, in which the σ_v and σ_h are the longitudinal and transverse stresses in the steel tube. The equivalent stress, σ_z , can be obtained from the following equation:

$$\sigma_z = \frac{\sqrt{2}}{2} \sqrt{(\sigma_v - \sigma_h)^2 + \sigma_v^2 + \sigma_h^2} \quad (2)$$

Since no load was applied directly to the steel tube, the longitudinal stresses in the tube adjacent to the gap are approximately zero. However, the longitudinal stresses at the mid-height of the specimens are of similar magnitude or even larger than the transverse stresses, with the stresses arising due to the bond stresses and frictional forces between the steel tube and the concrete core, as shown in Fig.12. This phenomenon happens not only for the unexposed specimens but also for the exposed specimens. Hence, although it is preferred for the tube to be only utilized to resist hoop strains (in the transverse direction) in order to maximize confinement, this is, in practice, difficult to achieve. The longitudinal and transverse stresses in the steel tube develop faster with increasing exposure time, resulting in earlier yielding of the steel tube. This can be explained by the significant reduction in stiffness of concrete with increased exposure times, leading to larger strains for a given load. For example, for specimens C250-30-30-b and C250-30-60-b, the steel tubes yielded when the applied load reached approximately 83% and 73% of their peak loads, respectively, while the steel tube yielded very near to the peak load for the unexposed specimen (C250-30-0-b).

Although the steel tube bears a certain degree of longitudinal stress at mid-height, no local buckling of the steel tubes was observed prior to the attainment of the peak load during testing, despite their diameter to thickness ratio (113.6) exceeding $90(235/f_y)$, the Class 3 slenderness limit specified by EC4 [20] for circular shaped CFST columns. This supports the feasibility of designing thinner encasing tubes, but with more longitudinal reinforcement in the concrete to enhance the fire performance of STCRC columns.

3. Finite element analysis

A 3D finite element model was developed using the program ABAQUS to investigate the fundamental structural behaviour of CSTCRC columns after fire. The transient temperature distributions in the CSTCRC columns, exposed to the ISO-834 standard fire, including the cooling phase, were calculated by means of a heat transfer analysis; the temperature results were then

imported into a subsequent stress analysis as a predefined field [21].

3.1 Material properties

For the heat transfer analysis, the thermal property models proposed by Lie [22] for steel and concrete were employed, including the density, heat conduction and specific heat. Previous studies [23,24] have shown that the mechanical properties of steel and concrete after fire exposure depend primarily on the maximum temperature attained during the heating and cooling phases. Hence, the post-fire stress-strain relationships for the materials can be obtained by substituting the key material factors after fire exposure (e.g., residual elastic modulus, ultimate stress and corresponding strain) for those of the unexposed material.

For the structural steel and reinforcing bars, the following elastic-perfectly plastic stress-strain relationship was adopted:

$$\sigma = \begin{cases} E_{sT} \varepsilon & \varepsilon \leq \varepsilon_{yT} \\ f_{yT} & \varepsilon > \varepsilon_{yT} \end{cases} \quad (3)$$

where the E_{sT} , f_{yT} and ε_{yT} are the residual elastic modulus, yield strength and yield strain, and $\varepsilon_{yT} = f_{yT} / E_{sT}$. For reinforcing bars, E_{sT} and f_{yT} should be replaced by E_{bT} and f_{bT} , respectively.

These parameters can be determined from [23] with the residual elastic modulus of structural steel given by:

$$E_{sT} = \begin{cases} E_s & T_{\max} \leq 500^\circ \text{C} \\ [1 - 1.30 \times 10^{-4} (T_{\max} - 500)] E_s & T_{\max} > 500^\circ \text{C} \end{cases} \quad (4)$$

The residual elastic modulus of reinforcing bars can be determined by substituting E_{bT} and E_b into Eq.(4).

The residual yield strength for structural steel is taken as:

$$f_{yT} = \begin{cases} f_y & T_{\max} \leq 500^\circ \text{C} \\ [1 - 2.33 \times 10^{-4} (T_{\max} - 500) - 3.88 \times 10^{-7} (T_{\max} - 500)^2] f_y & T_{\max} > 500^\circ \text{C} \end{cases} \quad (5)$$

and residual yield strength of the reinforcing bars is taken as:

$$f_{bT} = \begin{cases} f_b & T_{\max} \leq 500^\circ \text{C} \\ [1 - 5.82 \times 10^{-4} (T_{\max} - 500)] f_b & T_{\max} > 500^\circ \text{C} \end{cases} \quad (6)$$

where the E_s and f_y are the elastic modulus and the yield strength, respectively, of the structural steel at ambient temperature, E_b and f_b are the elastic modulus and the yield strength of reinforcing bars,

respectively, before fire exposure, and $T_{\max}$ is the maximum temperature ever achieved by the material.

The concrete model proposed by Han et al. [25], which has also been used by Yu et al. [26] for steel tube confined plain concrete columns, is used in this study, and given as follows:

$$y = \begin{cases} 2x - x^2 & x \leq 1 \\ \frac{x}{\beta_0(x-1)^2 + x} & x > 1 \end{cases} \quad (7)$$

in which $x = \varepsilon / \varepsilon_0$, $y = \sigma / \sigma_0$, $\xi = f_y A_s / f_{ck} A_c$, $\sigma_0 = f'_c$, $\varepsilon_0 = \varepsilon'_c + 800 \xi^{0.2} \times 10^{-6}$, $\varepsilon'_c = (1300 + 12.5 f'_c) \cdot 10^{-6}$, $\beta_0 = (2.36 \times 10^{-5})^{[0.25 + (\xi - 0.5)^7]} \cdot (f'_c)^{0.5} \cdot 0.5 \geq 0.12$, where A_s and A_c are the cross-sectional areas of the steel tube and the concrete, respectively, f'_c is the concrete cylinder strength and f_{ck} is the characteristic concrete strength and equals 0.67 times the concrete cube strength (f_{cu}). The initial elastic modulus E_c was taken as $4700 \sqrt{f'_c}$ N/mm², as recommended in the ACI specification [27].

The post-fire stress-strain relationship for the concrete can be obtained by substituting the post-fire elastic modulus, compressive strength and corresponding strain for those at ambient temperature, into Eq.(7). The post-fire properties, taken from [24], are as follows:

$$E_{cT} = E_c \frac{f'_{cT} / \varepsilon'_{cT}}{f'_c / \varepsilon'_c} \quad (8)$$

$$f'_{cT} = \frac{f'_c}{1 + 2.4(T_{\max} - 20)^6 \times 10^{-17}} \quad (9)$$

$$\varepsilon'_{cT} = \varepsilon'_c [1 + (1500 T_{\max} + 5 T_{\max}^2) \times 10^{-6}] \quad (10)$$

The elastic modulus in tension is assumed to be equal to that in compression. The tensile stress-strain relationship for the concrete is given as follows:

$$\sigma = \begin{cases} E_{cT} \varepsilon & \varepsilon \leq \varepsilon_{cr} \\ f'_{cT} \left(\frac{\varepsilon - \varepsilon_{tu}}{\varepsilon_{cr} - \varepsilon_{tu}} \right) & \varepsilon_{cr} < \varepsilon \leq \varepsilon_{tu} \\ 0 & \varepsilon > \varepsilon_{tu} \end{cases} \quad (11)$$

where

$$f'_{tT} = 0.1f'_{cT}, \quad \varepsilon_{cr} = \frac{f'_{tT}}{E_{cT}}, \quad \varepsilon_{tu} = 15\varepsilon_{cr}$$

The Poisson's ratio of the steel and concrete were assumed to be 0.3 and 0.2, respectively.

3.2 Heat transfer analysis

Transient heat transfer analyses were conducted to determine the temperature distribution in CSTCRC columns during the heating and cooling phases. The heat is transferred from the fire to the outer surface of the steel tube via radiation and convection, which is then transferred within the columns through conduction. The convective heat transfer coefficient and resultant emissivity were taken as 25W/(m²K) and 0.5 respectively. The thermal resistance at the interface between the steel tube and the concrete core was taken into account with a constant value of 100 W/(m²K) for the gap conductance [28,29], which has been shown to give acceptable predictions of temperature. The ambient temperature was defined to be 20 °C and the ISO-834 standard fire condition, including the cooling phase [12] was applied as a thermal load.

The three dimensional 8-noded continuum solid element (DC3D8) and 4-noded shell element (DS4) were used to model the concrete and steel tube, respectively. The reinforcing bars were modelled by means of 2-noded truss elements (DC1D2). The specific heat of the concrete was modified to take into account the influence of moisture [30]. It was assumed that all water would be vaporized at about 100 °C and that the content of water was 5% by weight.

3.3 Stress analysis

Nonlinear stress analyses were carried out subsequently to investigate the post-fire behaviour of CSTCRC columns. As discussed earlier, the post-fire properties of the steel and concrete depend on the maximum temperatures reached during the heating and cooling phases. However, clearly, the maximum temperature will not be achieved in the steel and concrete at the same time. Hence, the maximum nodal temperatures were extracted from the results of the heat transfer analysis with the user subroutine USDFLD and saved as a field variable. In order to read the nodal temperatures efficiently, the finite element meshes of the stress analysis were the same as those of the corresponding heat transfer analysis.

The concrete damage plasticity model implemented in ABAQUS was used for the infill concrete, which assumes that the main two failure mechanisms are tensile cracking and compressive crushing

of the concrete material. The classic metal plasticity model, with the Von Mises yield surface and associated plastic flow, was adopted for the steel tube and reinforcing bars.

The three-dimensional 8-noded solid element with reduced integration (C3D8R) and 4-noded shell element with reduced integration (S4R) were used to mesh the infill concrete and the steel tube, respectively. The reinforcing bars were modelled by means of 2-noded truss elements (T3D2).

The interaction between the steel tube and concrete core was modelled with the contact interaction algorithm in ABAQUS. The outer surface of the concrete and the inner surface of the steel tube were defined as a contact pair, of which the outer surface of concrete acts as the master surface and the inner surface of the steel tube acts as the slave surface. The interface behaviour in the normal direction was defined as ‘hard contact’, which allows for any contact pressure when surfaces are in contact and transmits no pressure when the surfaces separate. The interface behaviour in the tangential direction employed the Coulomb friction model, with a friction coefficient of 0.3. In this model, when the shear stress at the interface exceeds the bond strength, the surfaces begin to slide and the shear stress maintains a constant value, denoted as τ_{crit} , which can be defined as follows:

$$\tau_{\text{crit}} = \mu p \geq \tau_{\text{bond}} \quad (12)$$

where μ is the friction coefficient, p is the contact pressure between the steel tube and the concrete in the normal direction and τ_{bond} is the bond strength.

To date, research into the post-fire bond strength between a steel tube and its concrete infill is relatively scarce, and there are no standardised values available. Tao et al. [31] conducted experiments on the post-fire bond strength between these two elements in circular CFST columns, and found a lower limit of the bond strength to be 0.4 N/mm², independent of whether the member had been exposed to fire or not. Sensitivity analyses revealed that the variation in load bearing capacity is less than 5% for CSTCRC columns when the bond strength is varied from 0.4 N/mm² to unlimited. Therefore the value of $\tau_{\text{bond}}=0.4$ N/mm² is also assumed for the CSTCRC columns examined in this study, since CSTCRC columns share the same interface conditions and construction processes as CFST columns.

The embedded element technique was used to model the reinforcing bars.

3.4 Validation of the FE model

The FE model was validated by comparing its results with the experimental results on CSTCRC

columns generated in this paper. The comparisons are presented in Fig.8 and Fig.9, which shows good predictions of both temperatures and load-displacement curves.

In order to further validate the FE model, comparisons were made against additional existing tests on circular CFST columns tested after steady heating [32] and after exposure to the ISO-834 standard fire [33,34]. Details of these columns are given in Table 5 and Table 6. In the FE simulations, the bilinear stress-strain relationship used by Han [32,33] and Lin [34] was adopted to model the mechanical behaviour of the steel tube. For the CFST columns that followed the ISO-834 standard fire [33,34], the heating phase was controlled to follow the ISO-834 standard fire curve. However, the cooling phase was not specified. Therefore the delay of temperature rise cannot be considered in the simulation. Hence the temperatures of the specimens corresponding to the maximum furnace temperature were utilized to calculate the residual capacities as in [33,34]. Comparisons of the test and FE results, together with the FE results from the authors of the original studies, are shown in Fig.13 and Fig.14. Acceptable agreement may be seen, and it is concluded the FE model is able to predict accurately the post-fire behaviour of CFST columns.

4. Parametric studies and design method

Parametric studies were undertaken to extend the ranges of the key parameters to investigate their influence on the residual properties of CSTCRC columns after fire exposure. The considered parameters include heating time, cross-sectional dimensions, material strengths, steel tube to concrete area ratio and ratio of reinforcement. The values of these parameters are listed in Table 8.

The influences of these key parameters on the residual capacity of CSTCRC columns are shown in Fig.15. The residual capacities may be seen to decrease approximately linearly with increasing heating time, due to the deterioration of material properties at elevated temperatures, and the reduction in residual capacity is more severe for columns with smaller cross-sections, since, for a given exposure, the temperature in these columns is higher. The residual capacities increase approximately linearly with increasing material strength, steel tube to concrete area ratio and ratio of reinforcement.

The influences of the same parameters on the compressive stiffness are presented in Fig.16. Similar to the residual capacity, the compressive stiffness also decreases with increasing exposure time, but increases with higher cross-sectional diameters, concrete compressive strength and ratio of

reinforcement. The yield strengths of the steel tube and reinforcing bars and the steel tube to concrete area ratio had no clear influence on the compressive stiffness, and are therefore not presented.

Liu and Zhou [9] proposed a method for predicting the load bearing capacity of CSTCRC columns that have not been exposed to fire, given as follows:

$$N_u = f_{cc} A_c + f_b A_b \quad (13)$$

where A_c and A_b are the cross-sectional area of the concrete and longitudinal reinforcement, respectively, f_b is the yield strength of the reinforcement and f_{cc} is the compressive strength of the confined concrete, which can be determined as follows [35]:

$$f_{cc} = \left(-1.254 + 2.254 \sqrt{1 + 7.94 \frac{f_r}{f_c'}} - 2 \frac{f_r}{f_c'} \right) f_c' \quad (14)$$

where f_c' is the concrete cylinder strength and f_r is the confining stress provided by the steel tube to the concrete and can be calculated as follows:

$$f_r = \frac{2t_s f_y}{D - 2t_s} \quad (15)$$

Maintaining consistency with the above design method for columns at ambient temperature, a design method is proposed for evaluating the residual capacities of unstressed CSTCRC columns after exposure to the standard fire. The limits of validity of the design method are: $D=0.2 - 1.5$ m, $t_h=0 - 3$ h, $f_c'=20$ N/mm² - 50 N/mm², $f_y=235$ N/mm² - 420 N/mm², $f_b=335$ N/mm² - 500 N/mm², $\alpha_s=2\% - 4\%$, $\alpha_b=2\% - 6\%$. The formula is intended to predict the residual capacity of CSTCRC columns post ISO-834 fires in the studied parameter ranges, and may be used to determine an estimate of the post-fire residual capacity at the initial stage of assessment of fire damage.

$$N_{uT} = \bar{f}_{ccT} A_c + f_{bT} A_b \quad (16)$$

where

$$f_{bT} = \begin{cases} f_b & t_h \leq 1.0 \\ (1.075 - 0.075t_h) f_b & 1.0 < t_h \leq 3.0 \end{cases} \quad (17)$$

$$\bar{f}_{ccT} = \left(-1.254 + 2.254 \sqrt{1 + 7.94 \frac{f_{rT}}{\bar{f}_{cT}}} - 2 \frac{f_{rT}}{\bar{f}_{cT}} \right) \bar{f}_{cT} \quad (18)$$

$$\bar{f}_{cT}' = k \times [1 - (\frac{0.066}{D} - 0.008)t_h] f_c' \quad (19)$$

$$f_{rT} = \frac{2t_s f_{yT}}{D - 2t_s} \quad (20)$$

$$f_{yT} = (0.02t_h^2 - 0.15t_h + 1.0)f_y \quad (21)$$

where D is the cross-sectional diameter in metres, t_s is the wall thickness of the steel tube in metres, t_h is the heating time to the maximum temperature of the fire in hours, k is a parameter accounting for the effect of delay of temperature rise in the concrete, which is 0.98 and 1.0 for exposed and unexposed columns, respectively.

The experimental and numerical results reveal that the reduction of compressive stiffness after fire exposure is greater than the reduction in load bearing capacity. In order to evaluate deformations, a simplified formula for predicting the post-fire axial compressive stiffness is proposed as follows:

$$EA = \bar{E}_{cT} A_c + E_{bT} A_b \quad (22)$$

where

$$\bar{E}_{cT} = k \times [1 - (0.38 - 0.03t_h) \sqrt{\frac{t_h}{D}}] E_c \quad (23)$$

$$E_c = 4700 \sqrt{f'_c} \text{ MPa} \quad (24)$$

$$E_{bT} = \begin{cases} E_b & t_h \leq 1.0 \\ (1.015 - 0.015t_h) E_b & 1.0 < t_h \leq 3.0 \end{cases} \quad (25)$$

Comparisons of the predicted residual capacity and compressive stiffness using the design method and the FE model results are shown in Fig.17, which confirms the validity of the design method. The tested specimens in this paper were also predicted with the design method, as shown in Fig.18. Again, acceptable results can be obtained with the design method, though there are some discrepancies for the stiffness.

5. Conclusions

Experimental and numerical studies into the post-fire behaviour of circular steel tube confined reinforced concrete columns were conducted in this paper. Twenty one CSTCRC stub columns were tested to investigate their fundamental behaviour after exposure to the ISO-834 standard fire and to provide data for the validation of FE models. A sequentially coupled thermal-stress FE model was developed using the program ABAQUS, including heat transfer analysis and stress analysis. Having

validated the FE models against the tests conducted herein as well as by others, parametric studies were carried out, and on the basis of the results, a design method for predicting the residual capacity and compressive stiffness of CSTCRC columns after ISO-834 standard fire exposure was proposed. The following conclusions can be drawn based on these studies:

- (1) The maximum temperature attained during the heating and cooling phases decreases from the outer surface of the steel tube to the centre of the concrete, but the corresponding time increases markedly. The delay of temperature rise is attributed to the high thermal capacity of concrete and the protection from the surrounding material.
- (2) All specimens exhibited fairly ductile behaviour, though a shear failure mode was observed, regardless of whether or not the columns were exposed to fire. The residual capacity and compressive stiffness decrease with increasing heating time, with the compressive stiffness being more sensitive to exposure. The residual capacity and compressive stiffness were shown to increase with increasing cross-sectional dimensions, material strengths and steel tube to concrete area ratio.
- (3) A design method was proposed for predicting the residual capacity and compressive stiffness of CSTCRC columns after standard fire exposure. The method is intended to predict the residual properties of CSTCRC columns post ISO-834 fires in the studied parameter ranges, and may be used to provide a preliminary estimation of the post-fire residual capacity and stiffness at the initial stage of assessment of fire damage.

6. Acknowledgements

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7. References

- [1] Gardner NJ, Jacobson ER. Structural behavior of concrete filled steel tubes. ACI Journal Proceedings 1967; 64(7): 404-13.
- [2] Orito Y, Sato T, Tanaka N, Watanabe Y. Study on the unbonded steel tube concrete structure. In: Composite Construction in Steel and Concrete, ASCE 1987; 786-804.

- [3] Prion HGL, Boehme J. Beam-column behavior of steel tubes filled with high strength concrete. *Canadian Journal of Civil Engineering* 1994; 21: 207-18.
- [4] O' Shea MD, Bridge RQ. Tests on circular thin-walled steel tubes filled with very high strength concrete. University of Sydney, Department of Civil Engineering, Centre for Advanced Structural Engineering Research Report No. 754; 1997.
- [5] Fam A, Qie FS, Sami R. Concrete-filled steel tubes subjected to axial compression and lateral Cyclic Loads. *Journal of Structural Engineering, ASCE* 2004; 130(4): 631-40.
- [6] Tomii M, Sakino K, Watanabe K, Xiao Y. Lateral Load Capacity of Reinforced Concrete Short Columns Confined by Steel Tube. In: *Proceedings of the International Speciality Conference on Concrete Filled Steel Tubular Structures*, Harbin, China, 1985; 19-26.
- [7] Tomii M, Sakino K, Xiao Y, Watanabe K. Earthquake Resisting Hysteretic Behavior of Reinforced Concrete Short Columns Confined by Steel Tube. In: *Proceedings of the International Speciality Conference on Concrete Filled Steel Tubular Structures*, Harbin, China, 1985; 119-25.
- [8] Sakino K, Tomii M, Watanabe K. Sustaining Load Capacity of Plain Concrete Stub Columns Confined by Circular Steel Tube. In: *Proceeding of the International Speciality Conference on Concrete Filled Steel Tubular Structures*, Harbin, China, 1985; 112-8.
- [9] Liu JP, Zhou XH. Behavior and strength of tubed RC stub columns under axial compression. *Journal of Constructional Steel Research* 2010; 66: 28-36.
- [10] Zhou XH, Liu JP. Seismic behavior and shear strength of tubed RC short columns. *Journal of Constructional Steel Research* 2010; 66: 385-97.
- [11] Han LH, Qu H, Tao Z, Wang ZF. Experimental behaviour of thin-walled steel tube confined concrete column to RC beam joints under cyclic loading. *Thin-walled Structures* 2009; 47: 847-57.
- [12] ISO-834. Fire resistance tests -elements of building construction. Switzerland: International Organization for Standardization; 1975.
- [13] GB/T 228-2010. Metallic materials-Tensile testing-Part 1: Method of test at room temperature. Beijing: China Standard Press; 2010 [in Chinese].
- [14] GB/T 9978-2008. Test standard for fire resistance of construction members. Beijing: China Standard Press; 2008 [in Chinese].
- [15] Abrams MS. Compressive strength of concrete at temperatures to 1600 F. *ACI Special Publication* 1971; 25: 33-58.
- [16] Phan LT, Carino NJ. Review of mechanical properties of HSC at elevated temperature. *Journal of materials in Civil Engineering, ASCE* 1998; 10(1): 58-64.

- [17] Hertz KD. Concrete strength for fire safety design. Magazine of Concrete Research 2005; 57(8): 445-53.
- [18] Huo JS, Huang GW, Xiao Y. Effects of sustained axial load and cooling phase on post-fire behaviour of concrete-filled steel tubular stub columns. Journal of Constructional Steel Research 2009; 65: 1664-76.
- [19] Zhang SM, Guo LH, Ye ZL, Wang YY. Behavior of steel tube and confined high strength concrete for concrete-filled RHS tubes. Advances in Structural Engineering 2005; 8(6): 101-16.
- [20] EN 1994-1-1. Eurocode 4: Design of composite steel and concrete structures-part 1-1: General rules and rules for buildings, Brussel (Belgium), 2004.
- [21] SIMULIA. ABAQUS/Standard Version 6.9 analysis user's manual. Pawtucket, Rhode Island: Hibbit, Karlsson & Sorenson, Inc.; 2009.
- [22] Lie TT. Fire resistance of circular steel columns filled with bar-reinforced Concrete. Journal of Structural Engineering, ASCE 1994; 120(5): 1489-509.
- [23] Tao Z, Wang XQ, Uy B. Stress-strain curves of structural steel and reinforcing steel after exposure to elevated temperatures. Journal of Materials in Civil Engineering, ASCE 2013; 25(9): 1306-16.
- [24] Li W, Guo ZH. Experimental investigation of strength and deformation of concrete at elevated temperatures. Chinese Journal on Building Structures 1993; 14(1): 8-16 [in Chinese].
- [25] Han LH, Yao GH, Tao Z. Performance of concrete-filled thin-walled steel tubes under pure torsion. Thin-Walled Structures 2007; 45(1): 24-36.
- [26] Yu Q, Tao Z, Liu W, Chen ZB. Analysis and calculations of steel tube confined concrete (STCC) stub columns. Journal of Constructional Steel Research 2010; 66(1): 53-64.
- [27] ACI 318M-08. Building code requirements for structural concrete and Commentary. USA: American Concrete Institute; 2008.
- [28] Ding J, Wang YC. Realistic modelling of thermal and structural behaviour of unprotected concrete filled tubular columns in fire. Journal of Constructional Steel Research 2008; 64: 1086-102.
- [29] Lv XT, Yang H, Zhang SM. Effect of contact thermal resistance on temperature distributions of concrete-filled steel tubes in fire. Journal of Harbin Institute of Technology 2011; 18(1): 81-8.
- [30] Lu H, Zhao XL, Han LH. Finite element analysis of temperatures in concrete filled double skin steel tubes exposed to fires. In: 4th International structural engineering and construction conference, Melbourne, Australia, 2007; 1151-6.
- [31] Tao Z, Han LH, Uy B, Chen X. Post-fire bond between the steel tube and concrete in

- concrete-filled steel tubular columns. *Journal of Constructional Steel Research* 2011; 67(3): 484-96.
- [32] Han LH. Concrete-filled steel tubular structures-theory and practice. Beijing: China Science Press; 2007 [in Chinese].
- [33] Han LH, Huo JS, Wang YC. Compressive and flexural behaviour of concrete filled steel tubes after exposure to standard fire. *Journal of Constructional Steel Research* 2005; 61(7): 882-901.
- [34] Lin XK. Cyclic performance of concrete-filled steel tubular columns after exposure to fire. Dissertation for doctoral degree of Fuzhou University. 2006 [in Chinese].
- [35] Mander JB, Priestley MJN, Park R. Theoretical stress-strain model for confined concrete. *Journal of Structural Engineering, ASCE* 1988; 114(8): 1804-26.

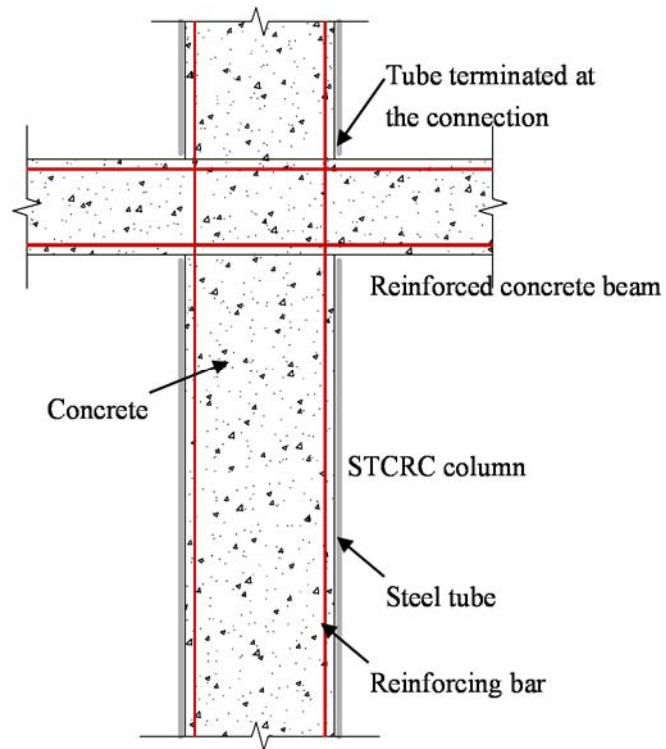
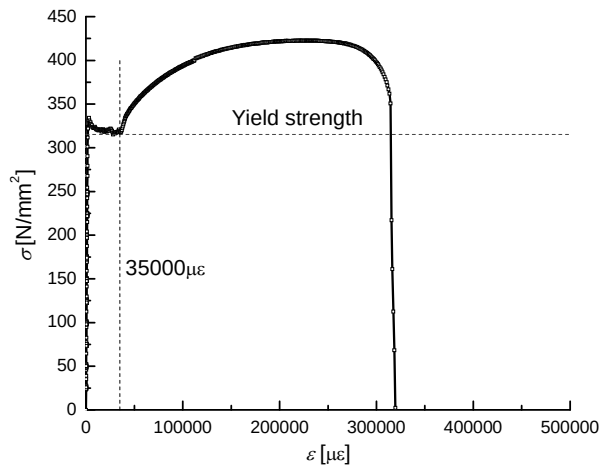
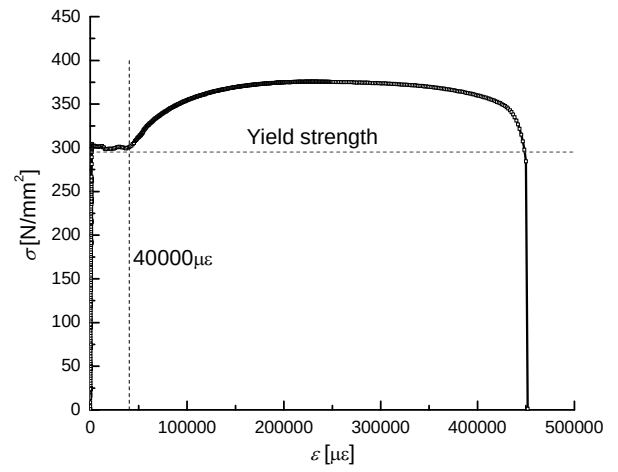


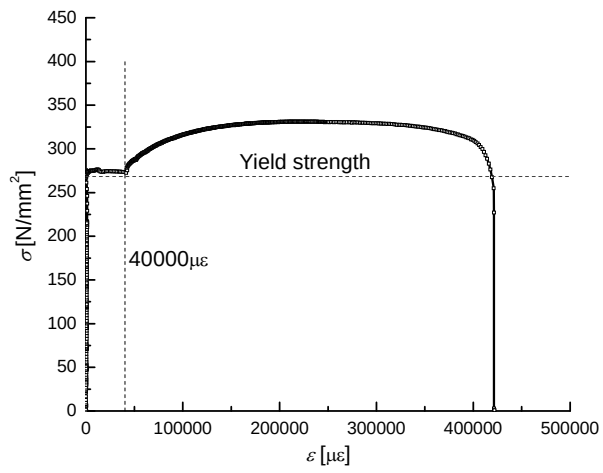
Fig.1 A schematic view of the STCRC column and its beam to column connection.



(a)

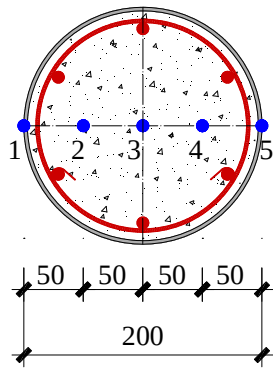


(b)

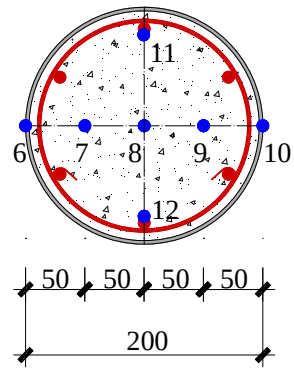


(c)

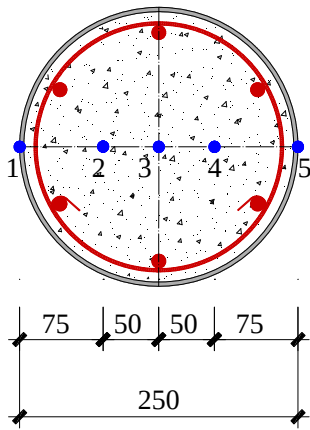
Fig.3 Stress-strain curves for steel tubes after heating times of: (a) 0 min; (b) 30 min; and (c) 60 min.



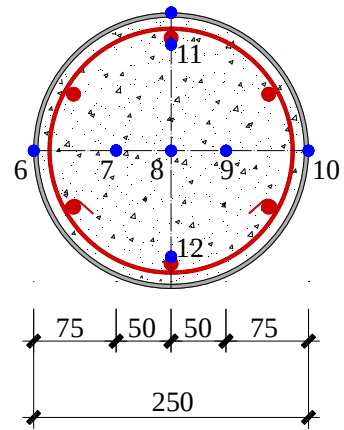
(a)



(b)



(c)



(d)

Fig.4 Layouts of thermocouples: C200-30min: (a) top; (b) bottom; C250-30min and C250-60min: (c) top; and (d) bottom (unit: mm).

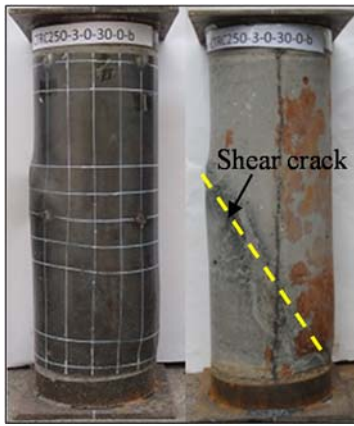


(a)

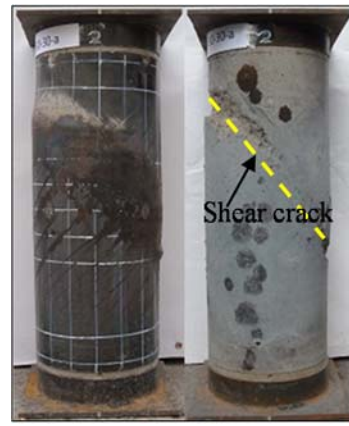


(b)

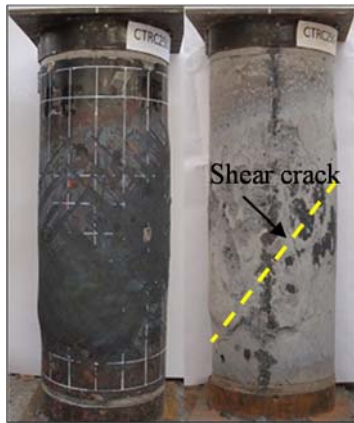
Fig.5 General view of the test set-up: (a) fire furnace; and (b) compression machine.



(a)

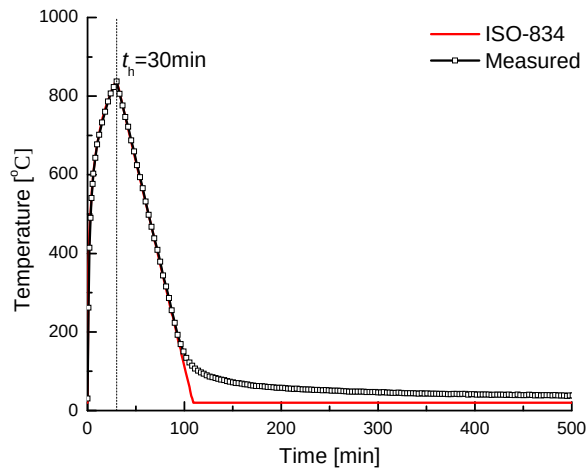


(b)

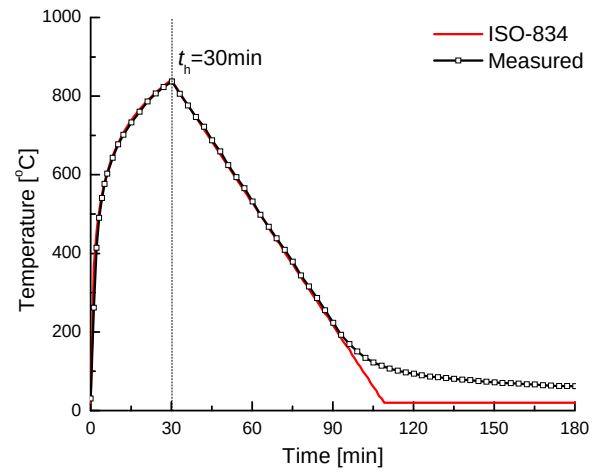


(c)

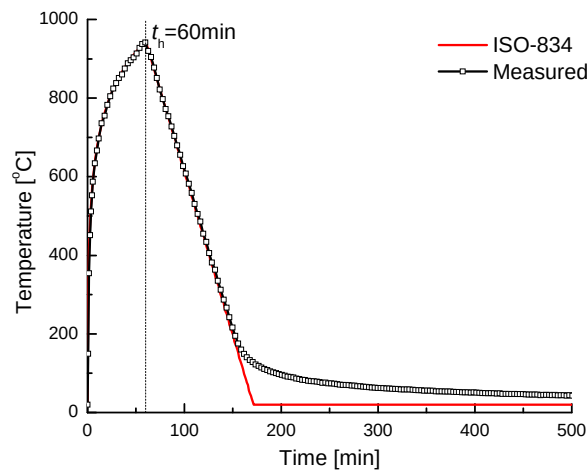
Fig.6 Typical failure modes of specimens: (a) C250-30-0; (b) C250-30-30; and (c) C250-30-60.



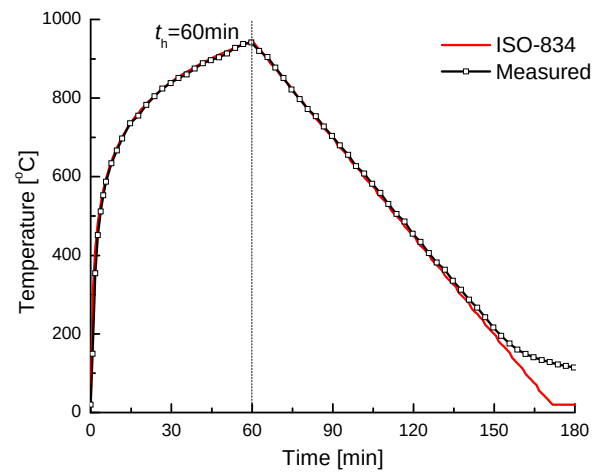
(a)



(b)

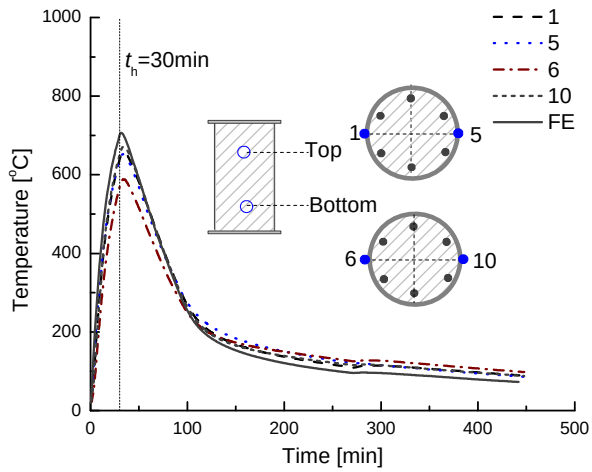


(c)

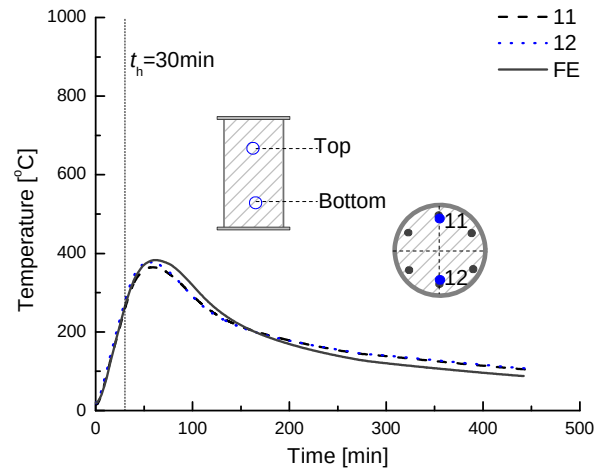


(d)

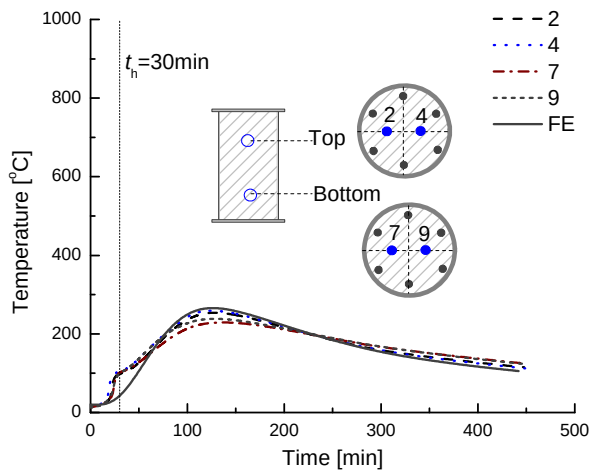
Fig.7 Comparisons of measured furnace temperature with ISO-834 standard fire curve: (a) 30min; (b) partial enlargement of (a); (c) 60min; and (d) partial enlargement of (c).



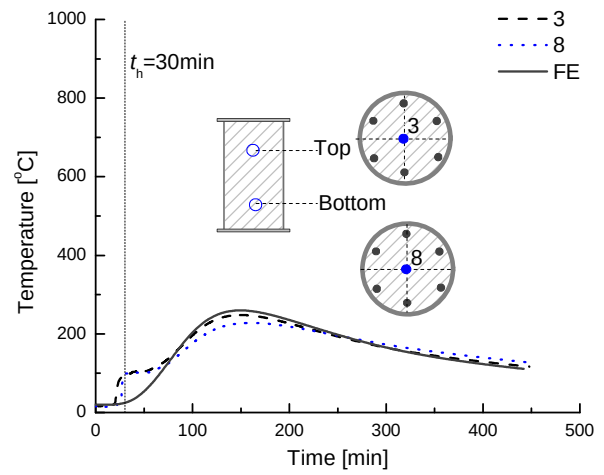
(a)



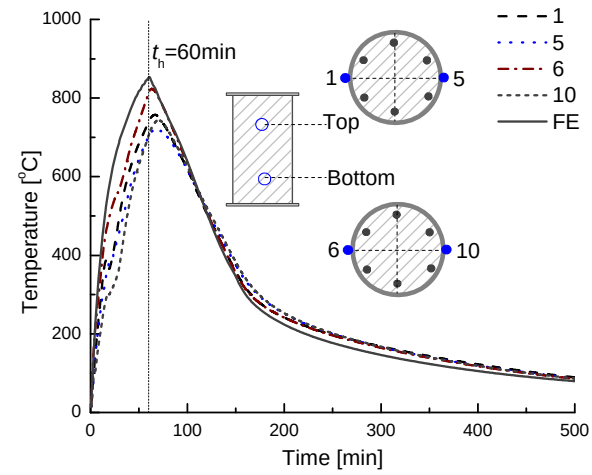
(b)



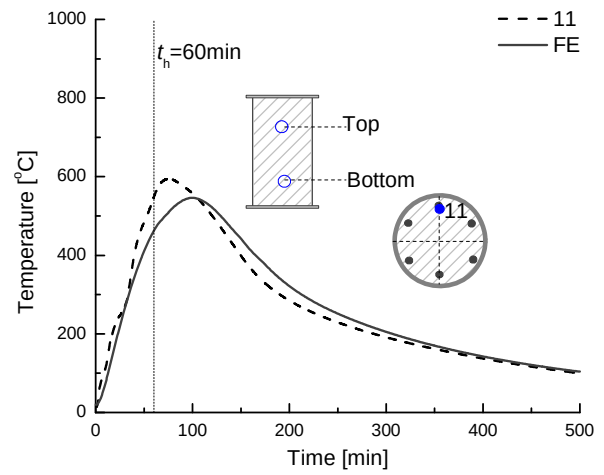
(c)



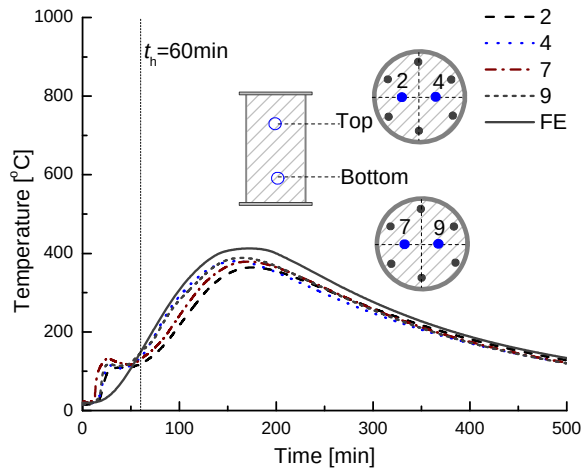
(d)



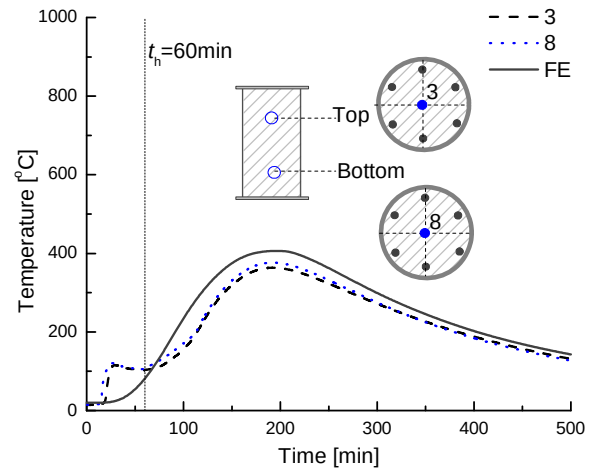
(e)



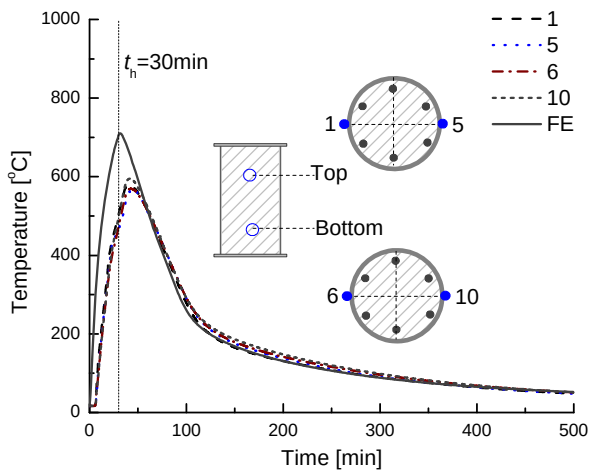
(f)



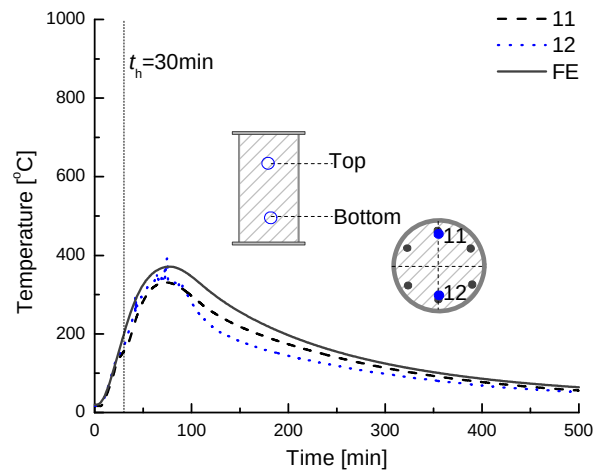
(g)



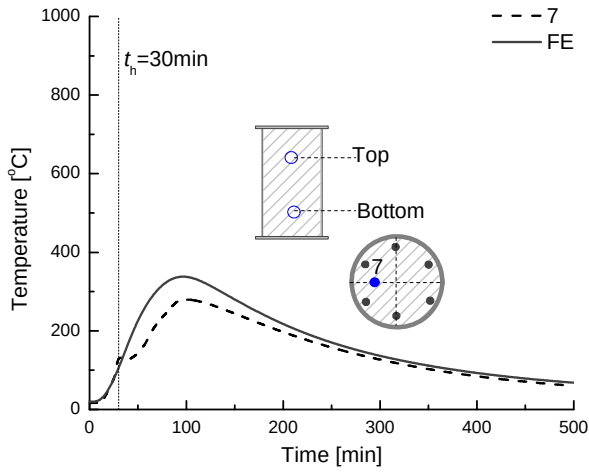
(h)



(i)

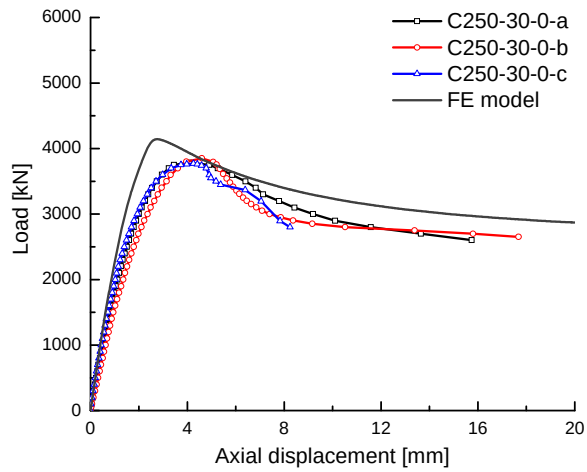


(j)

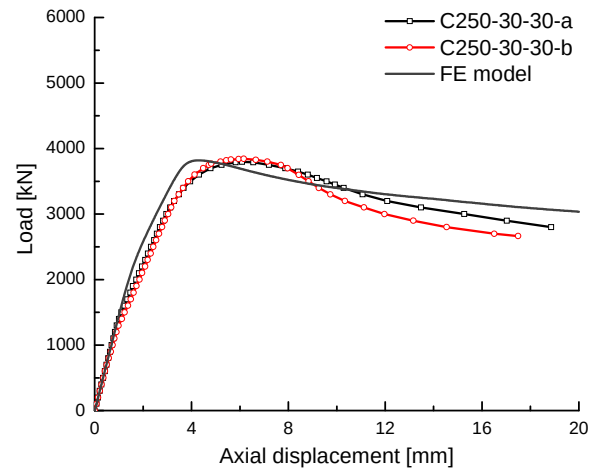


(k)

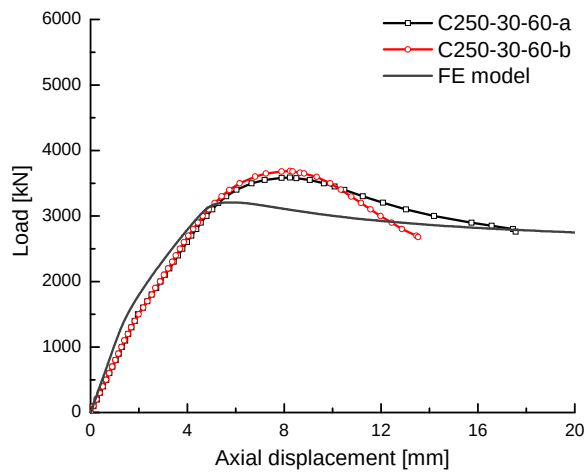
Fig.8 Measured and predicted cross-sectional temperatures of specimens: C250-30min: (a) steel tube; (b) reinforcement bars; (c) and (d) concrete; C250-60min: (e) steel tube; (f) reinforcement bars; (g) and (h) concrete; C200-30min: (i) steel tube; (j) reinforcement bars; (k) concrete.



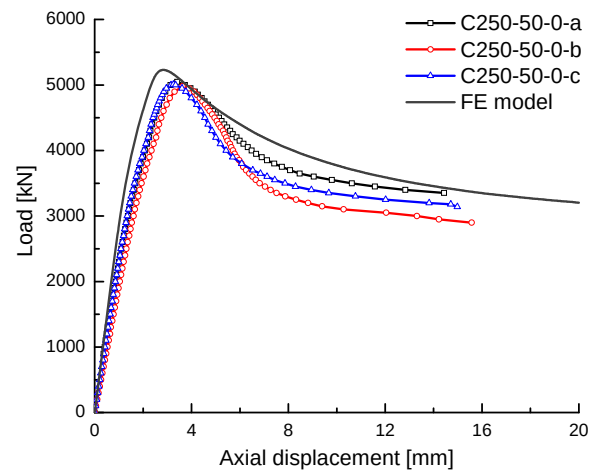
(a)



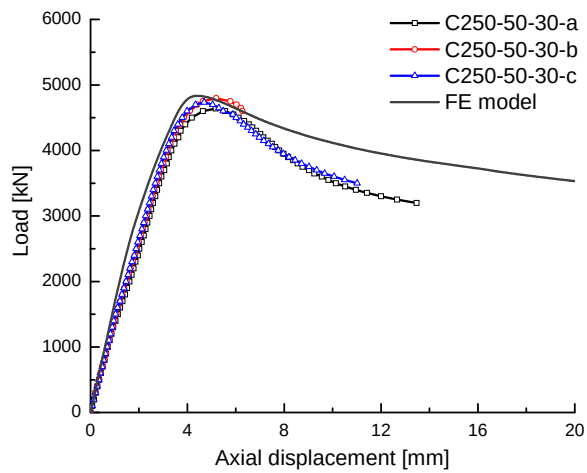
(b)



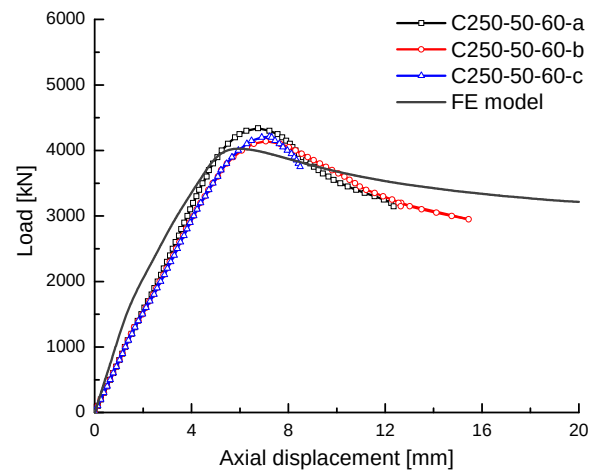
(c)



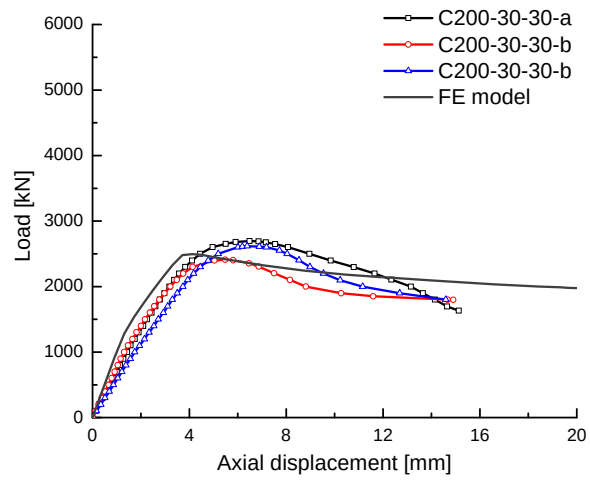
(d)



(e)



(f)



(g)

Fig.9 Axial load-displacement curves: (a) C250-30-0; (b) C250-30-30; (c) C250-30-60; (d) C250-50-0; (e) C250-50-30; (f) C250-50-60; and (g) C200-30-30.

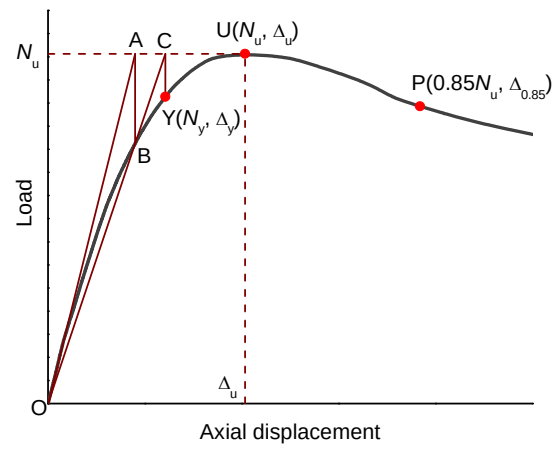
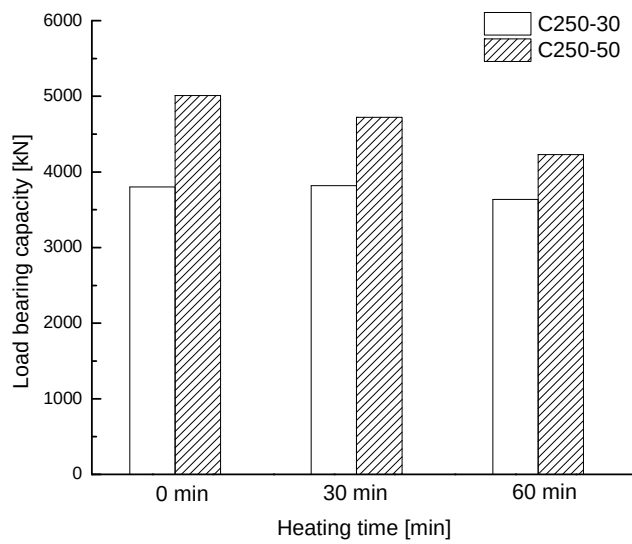
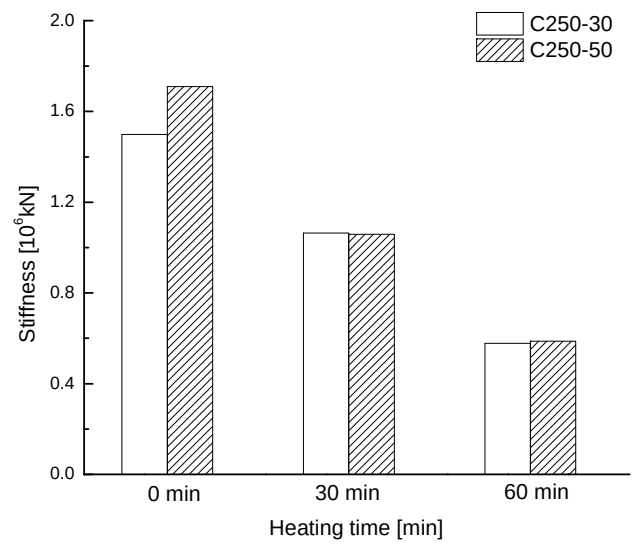


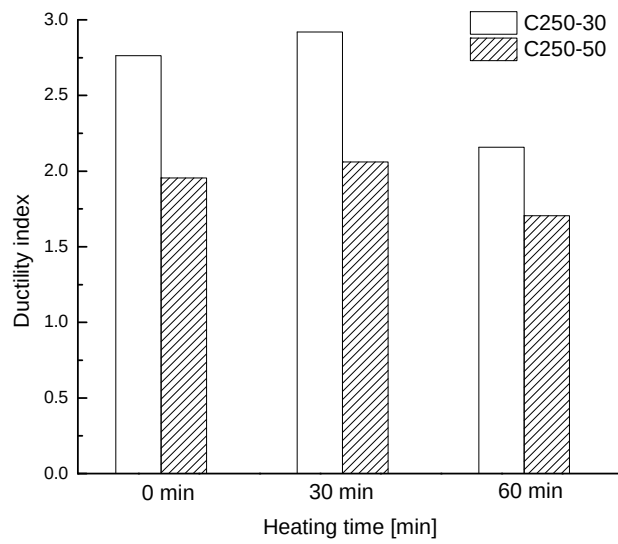
Fig.10 Determination of key stiffness and resistance, characteristics of test specimens



(a)

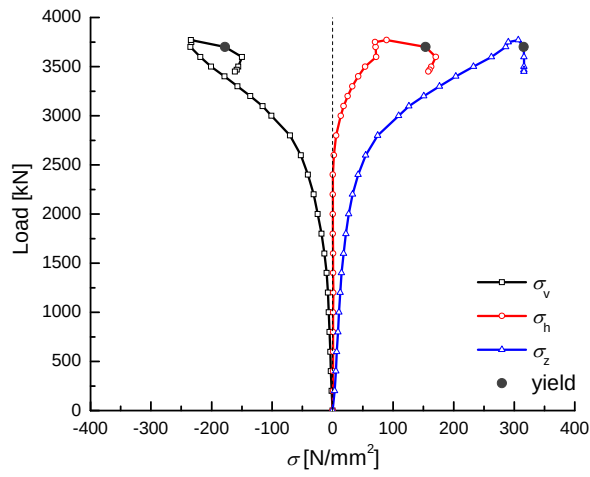


(b)

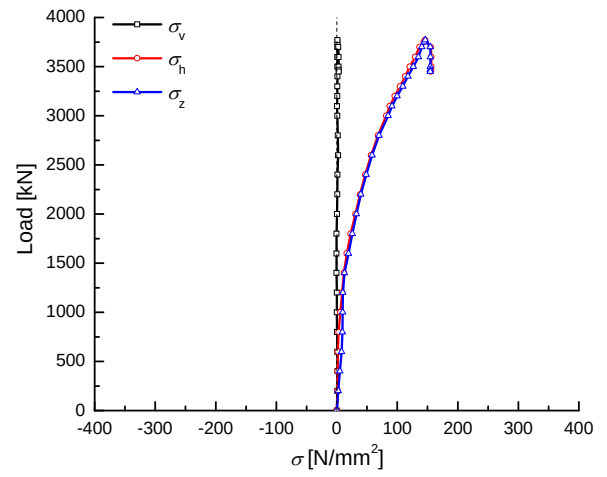


(c)

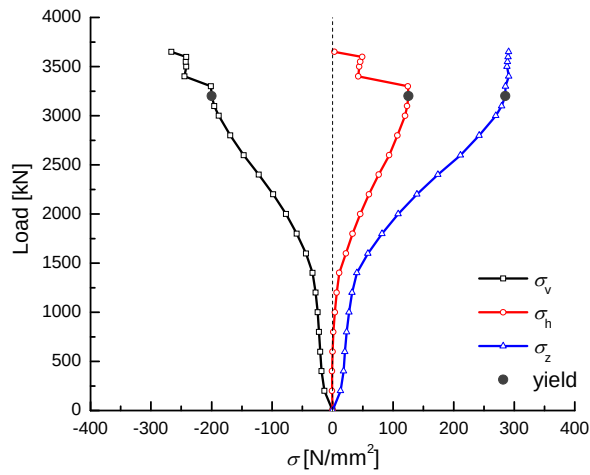
Fig.11 Influences of heating time and concrete strength on: (a) load bearing capacity; (b) stiffness; and (c) ductility index.



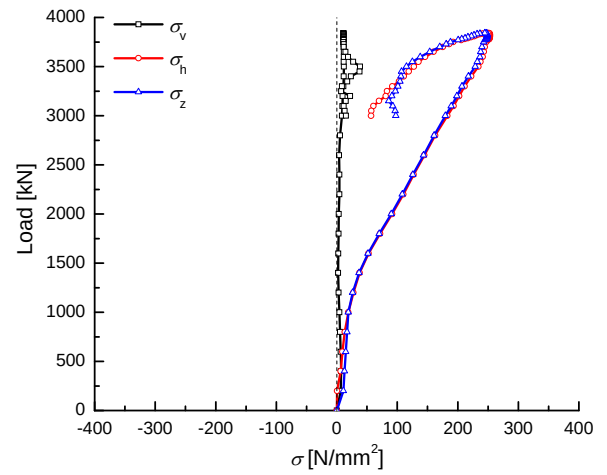
(a)



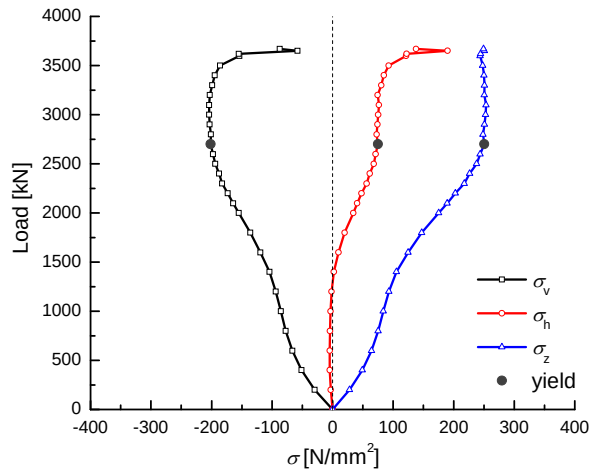
(b)



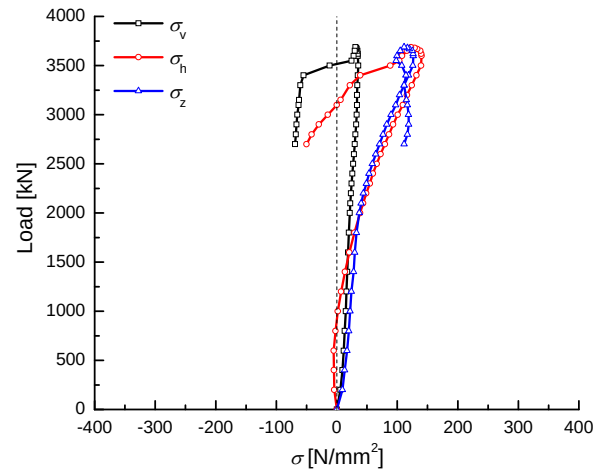
(c)



(d)

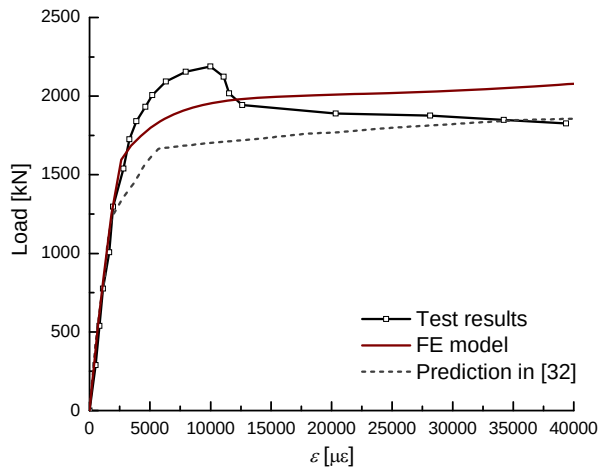


(e)

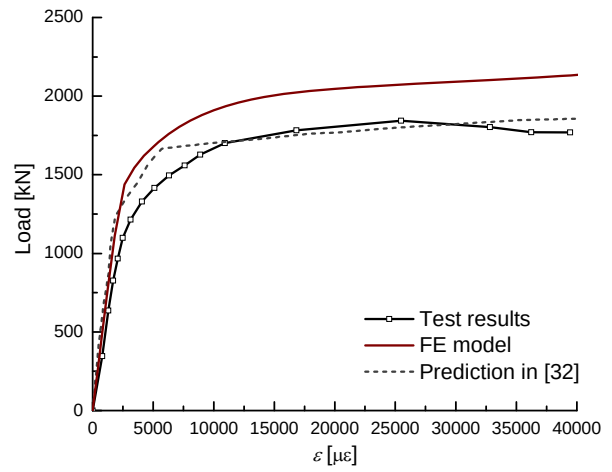


(f)

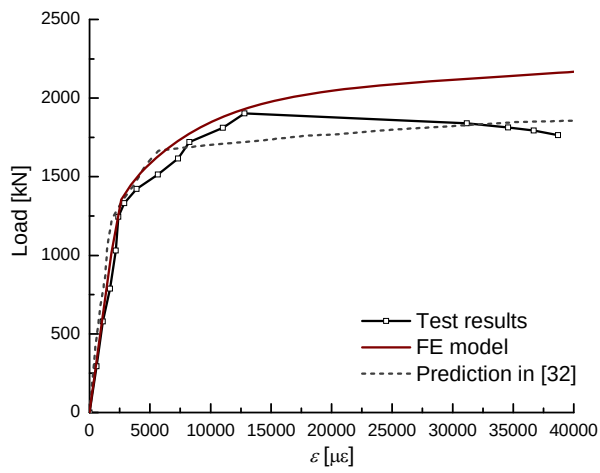
Fig.12 Axial load-steel tube stress curves: C250-30-0-b: (a) at mid-height; (b) edge of gap; C250-30-30-b: (c) at mid-height; (d) edge of gap; C250-30-60-b: (e) at mid-height; and (f) edge of gap.



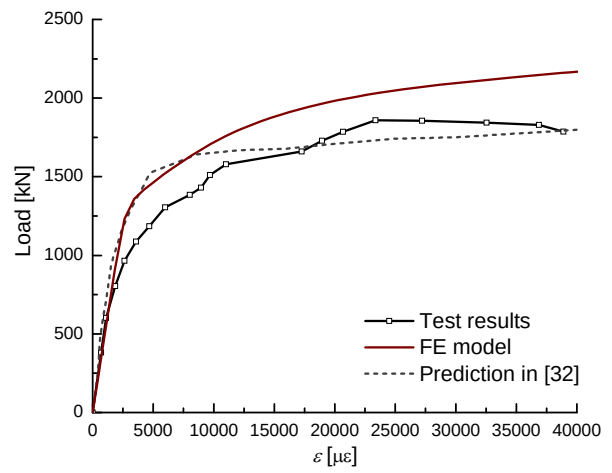
(a)



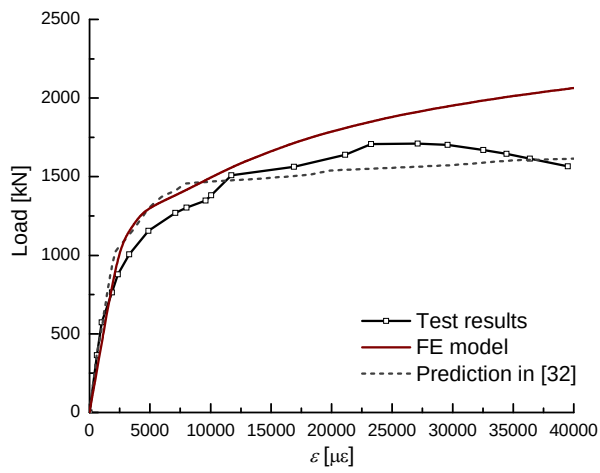
(b)



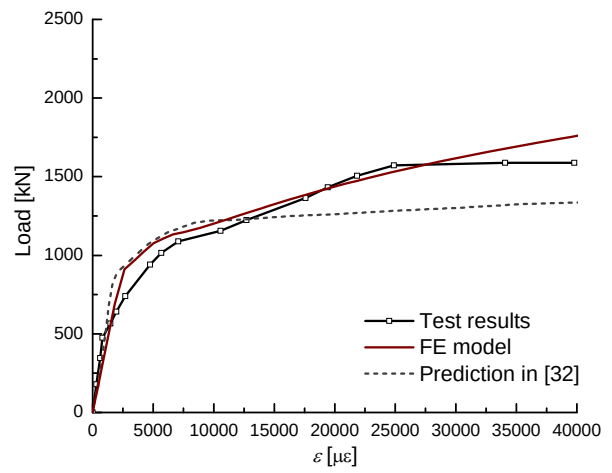
(c)



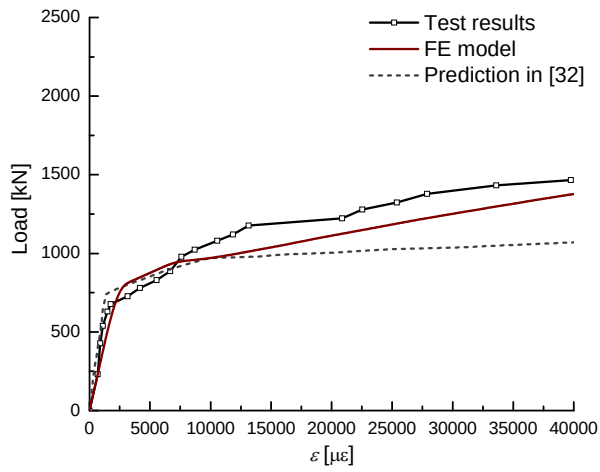
(d)



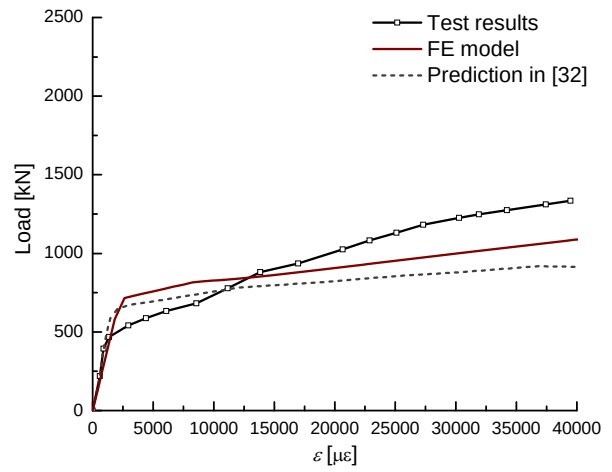
(e)



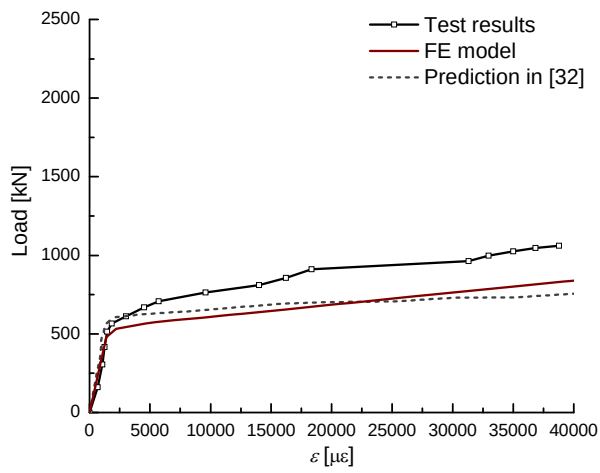
(f)



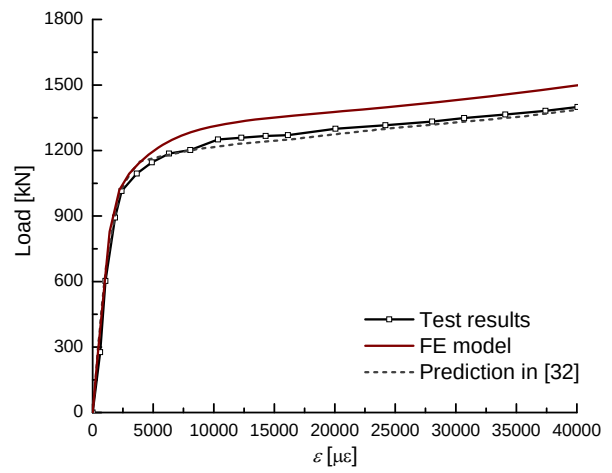
(g)



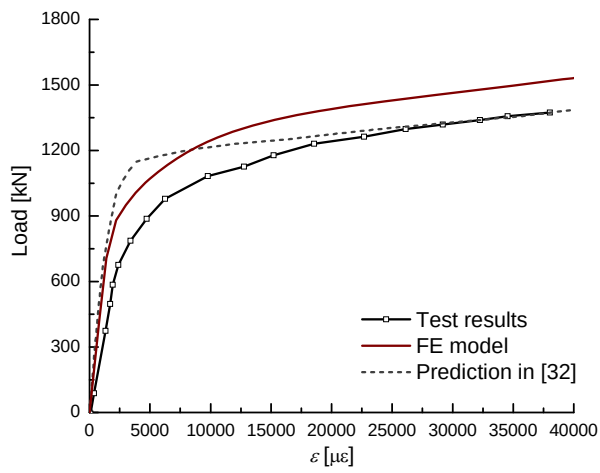
(h)



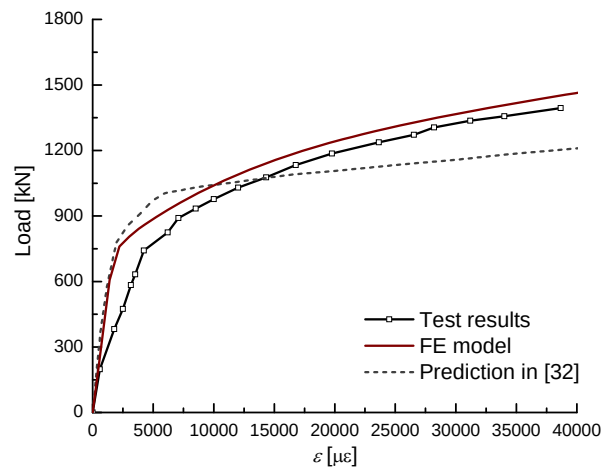
(i)



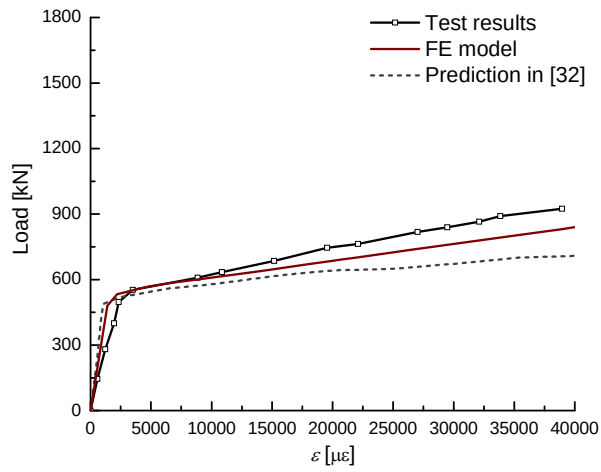
(j)



(k)

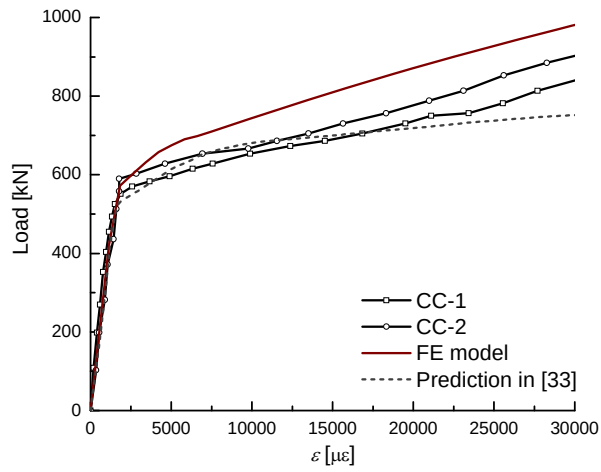


(m)

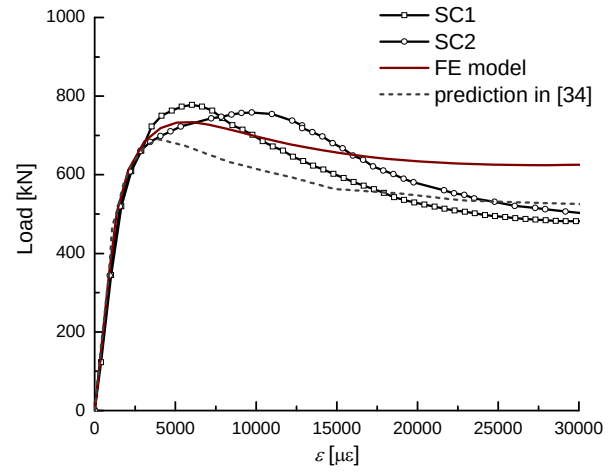


(n)

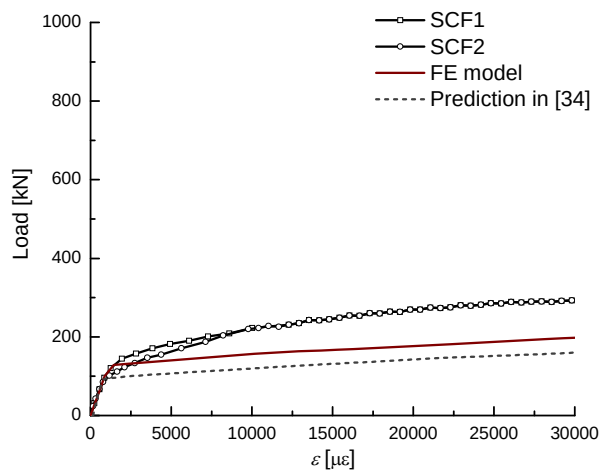
Fig.13 Comparisons of test and FE load-strain curves of CFST stub columns after steady heating [32]: (a) C1-20; (b) C1-200; (c) C1-300; (d) C1-400; (e) C1-500; (f) C1-600; (g) C1-700; (h) C1-800; (i) C1-900; (j) C2-20; (k) C2-300; (m) C2-500; and (n) C2-800.



(a)

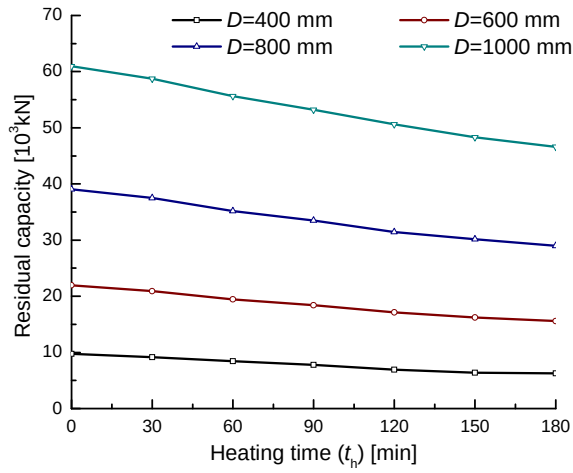


(b)

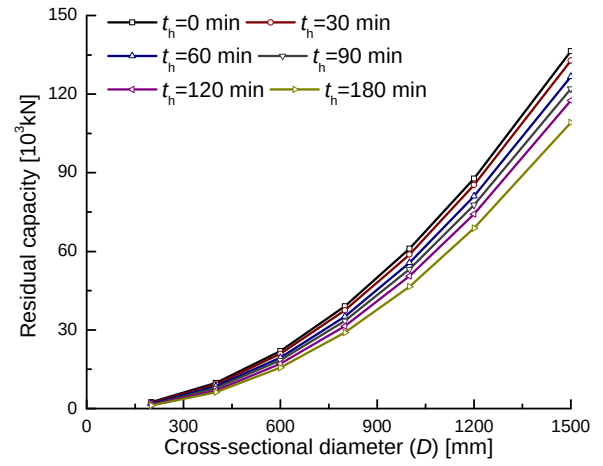


(c)

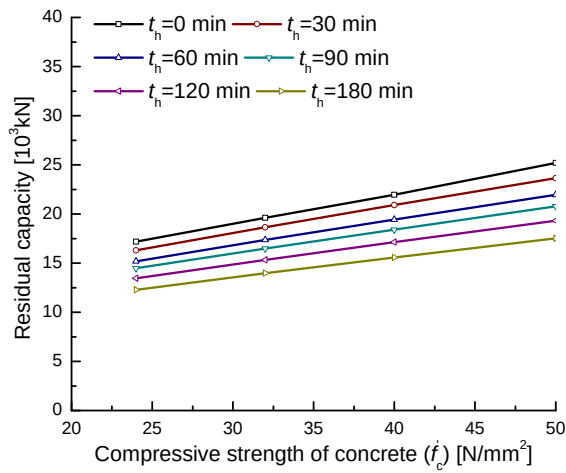
Fig.14 Comparisons of test and FE load-strain curves of CFST stub columns after standard fire: (a) CC-1 and CC-2 [33]; (b) SC1 and SC2 [34]; and (c) SCF1 and SCF2 [34].



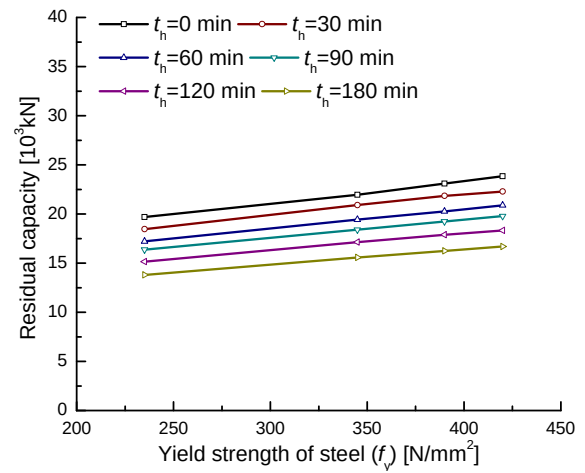
(a)



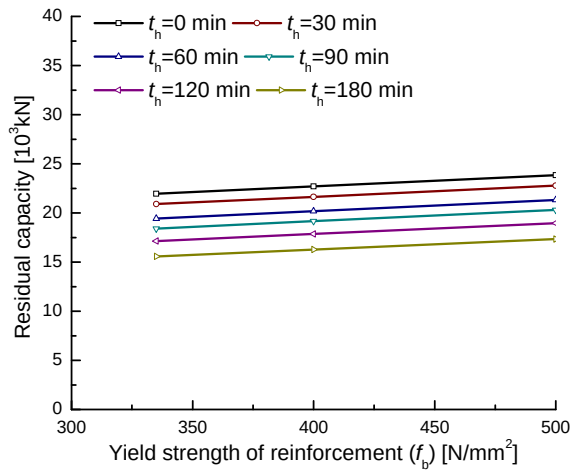
(b)



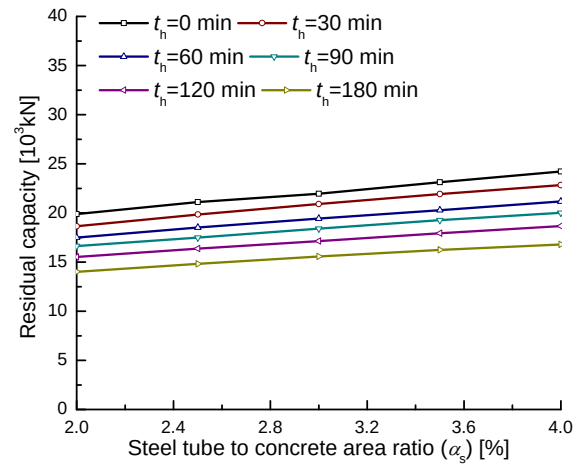
(c)



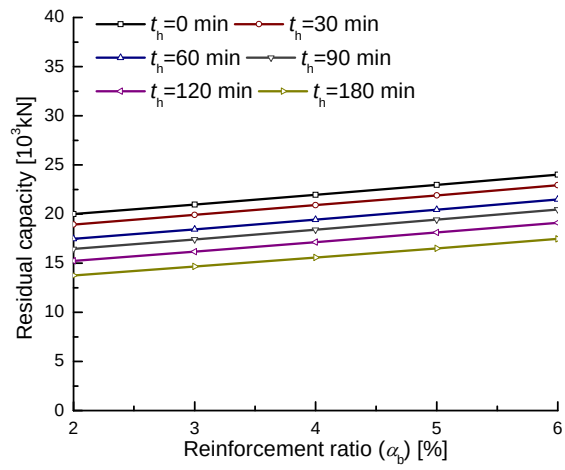
(d)



(e)

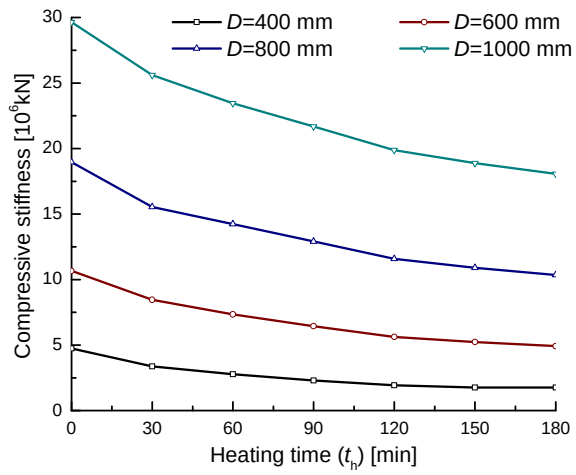


(f)

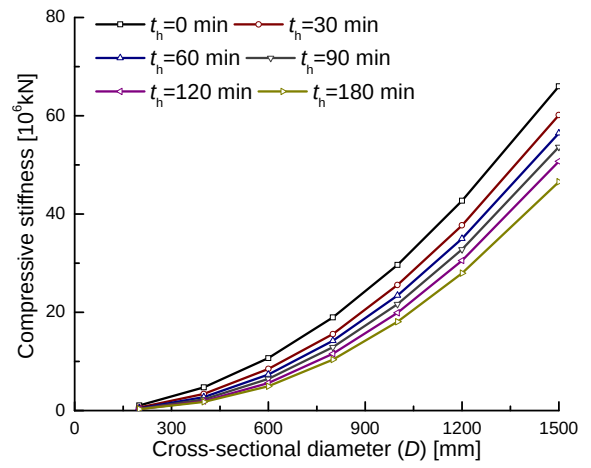


(g)

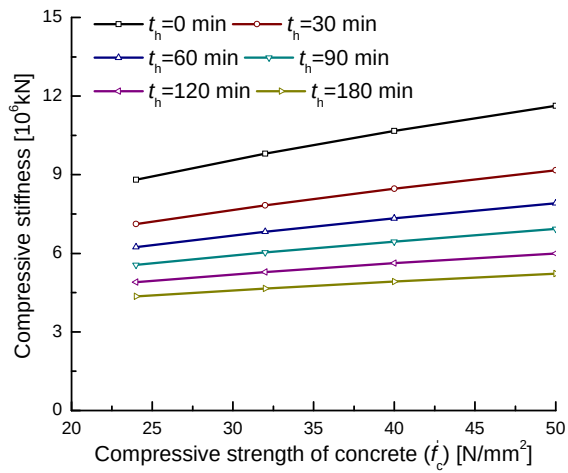
Fig.15 Influences of key parameters on residual capacities: (a) heating time; (b) cross-sectional dimension; (c) compressive strength of concrete; (d) yield strength of steel; (e) yield strength of reinforcement; (f) steel tube to concrete area ratio; and (g) reinforcement ratio.



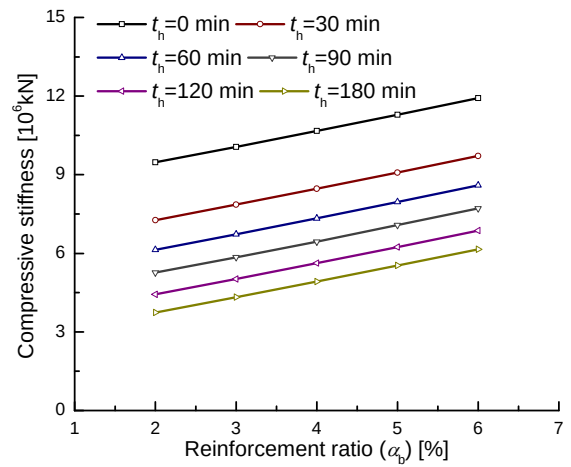
(a)



(b)

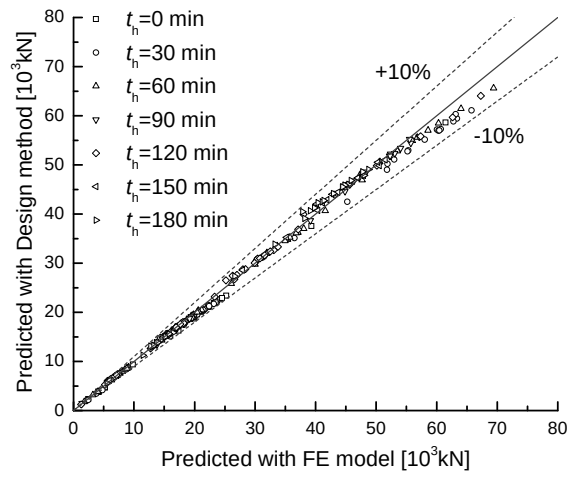


(c)

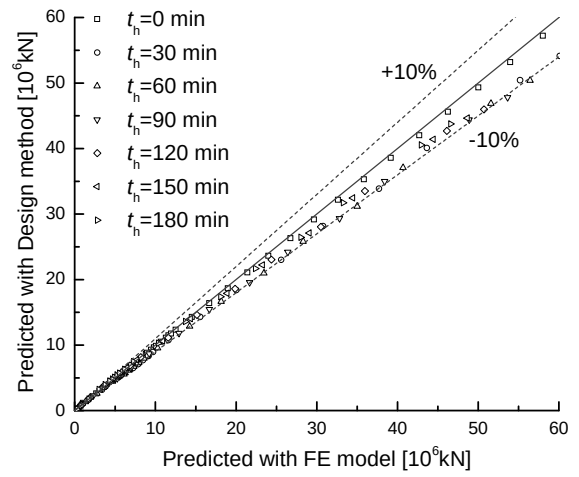


(d)

Fig.16 Influences of key parameters on compressive stiffness: (a) heating time; (b) cross-sectional dimension; (c) compressive strength of concrete; and (d) reinforcement ratio.

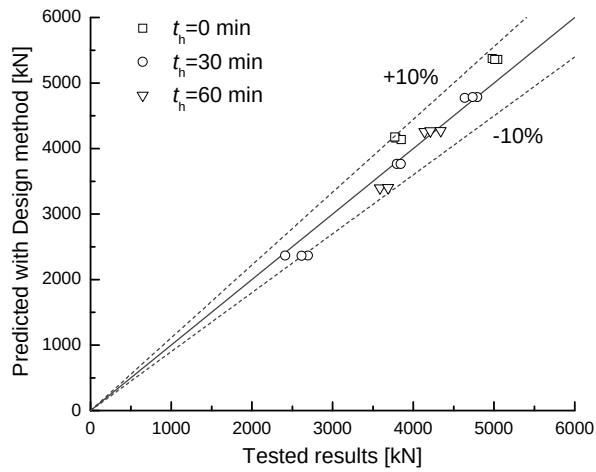


(a)

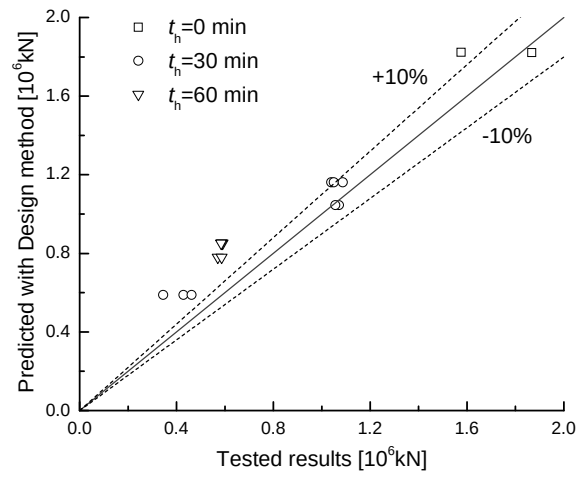


(b)

Fig.17 Comparisons of predicted residual capacity and compressive stiffness between the design method and FE model: (a) residual capacity; and (b) compressive stiffness.



(a)



(b)

Fig.18 Comparisons between predicted and tested results of specimens: (a) residual capacity; and (b) compressive stiffness.

Table 1 Details of test specimens

Column No.	D (mm)		t_s (mm)		α_s (%)	L (mm)	Reinforcing Bars	α_b (%)	t_h (min)
	Norminal	Measured	Norminal	Measured					
C250-30-0-a	250	249.0	2.20	2.11	3.62	750	6Φ20	3.98	0
C250-30-0-b	250	248.8	2.20	2.06	3.62	750	6Φ20	3.98	0
C250-30-0-c	250	249.5	2.20	2.14	3.62	750	6Φ20	3.98	0
C250-30-30-a	250	249.3	2.20	2.18	3.62	750	6Φ20	3.98	30
C250-30-30-b	250	249.7	2.20	2.18	3.62	750	6Φ20	3.98	30
C250-30-30-c	250	249.2	2.20	2.10	3.62	750	6Φ20	3.98	30
C250-30-60-a	250	249.5	2.20	2.18	3.62	750	6Φ20	3.98	60
C250-30-60-b	250	249.5	2.20	2.19	3.62	750	6Φ20	3.98	60
C250-30-60-c	250	249.7	2.20	2.17	3.62	750	6Φ20	3.98	60
C250-50-0-a	250	248.3	2.20	2.16	3.62	750	6Φ20	3.98	0
C250-50-0-b	250	249.8	2.20	2.19	3.62	750	6Φ20	3.98	0
C250-50-0-c	250	249.0	2.20	2.16	3.62	750	6Φ20	3.98	0
C250-50-30-a	250	248.7	2.20	2.16	3.62	750	6Φ20	3.98	30
C250-50-30-b	250	249.2	2.20	2.18	3.62	750	6Φ20	3.98	30
C250-50-30-c	250	249.2	2.20	2.18	3.62	750	6Φ20	3.98	30
C250-50-60-a	250	249.3	2.20	2.17	3.62	750	6Φ20	3.98	60
C250-50-60-b	250	249.0	2.20	2.14	3.62	750	6Φ20	3.98	60
C250-50-60-c	250	249.8	2.20	2.16	3.62	750	6Φ20	3.98	60
C200-30-30-a	200	199.7	1.76	1.73	3.62	600	6Φ16	3.98	30
C200-30-30-b	200	200.2	1.76	1.73	3.62	600	6Φ16	3.98	30
C200-30-30-c	200	199.4	1.76	1.72	3.62	600	6Φ16	3.98	30

Table 2 Ambient temperature properties of steel tube after fire exposure times of 0, 30 and 60 minutes

Norminal t_s (mm)	t_h (min)	E_s (N/mm ²)	f_y (N/mm ²)	f_{su} (N/mm ²)	ν_s	ε_{sf} (%)
1.76	0	1.99×10^5	342.9	488.8	0.263	27.44
2.20	0	1.98×10^5	318.9	424.9	0.304	35.31
1.76	30	1.87×10^5	341.5	445.7	0.292	37.93
2.20	30	2.07×10^5	295.2	378.1	0.294	48.69
2.20	60	1.82×10^5	262.1	327.9	0.281	44.50

Table 3 Properties of longitudinal reinforcing bars and stirrups

Steel type	Norminal d_s (mm)	E_b (N/mm ²)	f_b (N/mm ²)	f_{bu} (N/mm ²)	ε_{bf} (%)
Hot-rolled plain	8.0	1.93×10^5	435.4	668.8	17.71
Hot-rolled ribbed	16.0	1.64×10^5	326.2	498.0	24.73
Hot-rolled ribbed	20.0	1.84×10^5	357.4	570.5	20.00

Table 4 Concrete cube strength and elastic modulus

Nominal f_{cu} (N/mm ²)	$f_{cu,28}$ (N/mm ²)	$E_{c,28}$ (N/mm ²)	$f_{cu,test}$ (N/mm ²)	$E_{c,test}$ (N/mm ²)	Age of concrete at test day (days)
30	33.7	2.52×10^4	53.8	-	184
50	63.6	3.63×10^4	76.3	3.72×10^4	289

Table 5 Experimental results of CSTRC columns

Group No.	Column No.	EA (10 ⁶ kN)	N_y (kN)	Δ_y (mm)	N_u (kN)	Δ_u (mm)	$\Delta_{0.85}$ (mm)	μ_Δ
C250-30	C250-30-0-a	1.576	3316	2.42	3783	4.07	7.68	3.17
	C250-30-0-b	1.053	3503	3.44	3851	4.89	6.54	1.90
	C250-30-0-c	1.868	3192	2.18	3770	4.25	7.01	3.21
	Averaged	1.499	3337	2.68	3801	4.40	7.08	2.76
	C250-30-30-a	1.071	3392	3.66	3798	6.08	11.80	3.22
	C250-30-30-b	1.056	3471	3.80	3843	6.16	9.94	2.62
	C250-30-30-c	NA	NA	NA	NA	NA	NA	NA
	Averaged	1.064	3432	3.73	3820	6.12	10.87	2.92
	C250-30-60-a	0.570	3318	6.00	3586	8.56	13.95	2.32
	C250-30-60-b	0.586	3401	5.74	3688	8.25	11.43	1.99
	C250-30-60-c	NA	NA	NA	NA	NA	NA	NA
	Averaged	0.578	3359	5.87	3637	8.41	12.69	2.16
C250-50	C250-50-0-a	1.843	4666	2.62	5047	3.41	5.62	2.15
	C250-50-0-b	1.500	4704	2.98	4979	3.64	5.41	1.81
	C250-50-0-c	1.786	4672	2.58	5008	3.25	4.92	1.91
	Averaged	1.710	4681	2.73	5011	3.43	5.32	1.96
	C250-50-30-a	1.040	4381	3.88	4639	5.17	8.01	2.07
	C250-50-30-b	1.050	4527	3.93	4795	5.20	NA	NA
	C250-50-30-c	1.087	4481	3.75	4735	4.70	7.64	2.04
	Averaged	1.059	4463	3.85	4723	5.02	7.83	2.06
	C250-50-60-a	0.590	4210	5.83	4340	6.75	9.29	1.59
	C250-50-60-b	0.587	3941	5.87	4139	7.06	10.67	1.82
	C250-50-60-c	0.584	4023	6.02	4212	7.22	NA	NA
	Averaged	0.587	4058	5.91	4230	7.01	9.98	1.71
C200-30	C200-30-30-a	0.429	2505	4.47	2696	6.48	10.84	2.43
	C200-30-30-b	0.462	2223	3.84	2412	5.48	8.50	2.21
	C200-30-30-c	0.345	2480	5.11	2617	6.39	9.41	1.84
	Averaged	0.412	2403	4.47	2575	6.12	9.58	2.16

Table 6 Circular CFST stub columns after steady heating [32]

Column No.	D (mm)	t_s (mm)	L (mm)	T (°C)	f_y (N/mm ²)	f_{cu} (N/mm ²)
C1-20	133	4.5	399	20	433	72.4
C1-200	133	4.5	399	200	433	72.4
C1-300	133	4.5	399	300	433	72.4
C1-400	133	4.5	399	400	433	72.4
C1-500	133	4.5	399	500	433	72.4
C1-600	133	4.5	399	600	433	72.4
C1-700	133	4.5	399	700	433	72.4
C1-800	133	4.5	399	800	433	72.4
C1-900	133	4.5	399	900	433	72.4
C2-20	133	4.5	399	20	324	40.8
C2-300	133	4.5	399	300	324	40.8
C2-500	133	4.5	399	500	324	40.8
C2-800	133	4.5	399	800	324	40.8

Table 7 Circular CFST stub columns after ISO-834 standard fire [33,34]

Column No.	D (mm)	t_s (mm)	L (mm)	t_h (min)	f_y (N/mm ²)	f_{cu} (N/mm ²)
CC-1[33]	133	4.8	400	90	340	53.2
CC-2[33]	133	4.8	400	90	340	53.2
SC1[34]	100	2	300	0	290	75
SC2[34]	100	2	300	0	290	75
SCF1[34]	100	2	300	180	290	75
SCF2[34]	100	2	300	180	290	75

Table 8 Values of studied parameters

Parameter	Values	Fixed value
t_h (min)	0, 30, 60, 90, 120, 150, 180	-
D (mm)	200, 400, 600, 800, 1000, 1200, 1500	600
f'_c (N/mm ²)	24, 32, 40, 50	40
f_y (N/mm ²)	235, 345, 390, 420	345
f_b (N/mm ²)	335, 400, 500	335
α_s (%)	2.0, 2.5, 3.0, 3.5, 4.0	3.0
α_b (%)	2.0, 3.0, 4.0, 5.0, 6.0	4.0