Experimental and numerical studies of reinforced concrete columns confined by circular steel tubes exposed to fire

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Abstract:

Reinforced concrete columns confined by steel tubes, also known as steel tube confined reinforced concrete (STCRC) columns, are a kind of composite column in which the outer steel tube acts predominantly as hoop reinforcement. This is achieved by the provision of breaks to the longitudinal continuity of the steel tube. The compressive behavior and seismic performance of STCRC columns have been extensively studied in the past few decades. However, knowledge of the fire behavior of STCRC columns is very limited. Hence, experimental and numerical studies to investigate the response of STCRC columns under combined thermal (fire) and structural loading are presented herein. Four full-scale STCRC columns and one concrete-filled steel tubular (CFST) column were first axially loaded and then subjected to the ISO 834 standard fire until failure. The measured furnace temperatures, specimen temperatures, axial displacement versus time curves and fire resistance of the columns are presented and discussed. A nonlinear finite element model, employing a sequentially coupled thermal-stress analysis, was then developed in ABAQUS and validated against recent fire tests on STCRC columns and concrete-filled steel tubular (CFST) columns reported in the literature. Following extensive parametric studies, a simplified method is proposed for predicting the temperatures of the steel tube, the reinforcing bars and the concrete. Design rules are then proposed for predicting the load-bearing capacity of STCRC columns exposed to the ISO 834 standard fire,

which are consistent with the design method for STCRC columns at ambient temperature.

Key words: experiments, fire resistance, finite element, steel tube confined reinforced concrete, temperature, design method

1 **Introduction**

2 Steel tube confined reinforced concrete (STCRC) columns are composite columns where the outer
3 steel tube serves principally as hoop reinforcement. This is achieved by breaking the longitudinal
4 continuity of the steel tube (i.e. introducing gaps). In this arrangement, the confinement effect to the
5 concrete occurs from the onset of loading and can be maximized, which greatly enhances the
6 strength and ductility of the concrete. Local buckling of the steel tube in STCRC columns is
7 generally prevented or at least delayed compared to that in concrete-filled steel tubular (CFST)
8 columns, since no vertical load is directly applied to the tube; this allows much thinner steel tubes to
9 be used in STCRC columns than in CFST columns. In order to prevent load being applied to the tube
10 directly, the steel tube is usually terminated at the beam to column connections, rather than being
11 continuous. STCRC column to reinforced concrete beam connections are therefore similar to
12 connections in reinforced concrete structures, and can be designed and constructed following
13 established methods for conventional reinforced concrete.

14 Numerous studies have been conducted to date on the compressive behavior and seismic
15 performance of this kind of column (Tommi et al. 1985a, 1985b; Sakino et al. 1985; Aboutaha et al.
16 1998, 1999; Sun and Sakino 2000; Sun and Fukuhara 2005; Han et al. 2005a, 2009; Liu and Zhou
17 2010; Liu et al. 2015; Wang et al. 2015; Zhou et al. 2016; Wang and Liu 2017). However, the
18 behavior of circular STCRC columns at elevated temperatures has received less attention. Hence, the
19 fire and post-fire behavior of STCRC columns is the focus of ongoing research by the authors. The
20 post-fire response of STCRC columns, including their cross-sectional behavior, buckling behavior
21 and behavior under combined compression and bending, were experimentally and numerically
22 studied by the authors and presented by Liu et al. (2014), Yang et al. (2015) and Liu et al. (2016),
23 respectively. The present paper focuses on the performance of STCRC columns subjected to standard
24 fire conditions, in order to evaluate the fundamental behavior of these columns in fire and to obtain
25 essential information to underpin their fire safety design. STCRC columns resemble to CFST
26 columns in appearance, and research methods of fire performance of CFST columns can be referred
27 to. To date, extensive studies on fire performance of CFST columns have been conducted, especially
28 by research groups led by Lie and Kodur (Lie and Chabot 1992; Chabot and Lie 1992; Lie 1994; Lie

29 and Kodur 1996; Kodur and Lie 1996; Kodur 1999; Schaumann et al. 2009) and Han (Han et al.
30 2003a; 2003b; 2003c; 2005b; Yang et al. 2008; Han et al. 2012). Amongst them, constitutive model
31 for the confined core concrete under/after elevated temperatures with the “confinement factor” as an
32 essential variable (Han 2007; Yang et al 2008) was established, which provides an essential base for
33 the investigations of full-range fire exposed concrete-filled steel tubular columns. From the above
34 research onwards, the research on fire performance of CFST columns has been further continued
35 (Ding and Wang 2008; Espinos et al. 2010; Espinos et al. 2011; Hong and Varma 2009; Ibáñez et al.
36 2013; Tao et al. 2012, 2016; Neuenschwander et al. 2017a; Xiong and Liew 2016; Yang et al. 2013a,
37 2013b; Yu et al. 2018).

38 Fire tests were first conducted to investigate the behavior of circular STCRC columns subjected to
39 the ISO 834 standard fire. Five full-scale specimens, including four STCRC columns and one CFST
40 column, were tested to failure under combined thermal and structural loading. The furnace and
41 specimen temperatures, the axial deformation versus time curves, the fire resistance and the failure
42 modes of the specimens were all recorded during the tests. Following testing, an advanced finite
43 element (FE) model was developed using the program ABAQUS, utilizing a sequentially coupled
44 thermal-stress analysis, to investigate the fire performance of circular STCRC columns. The FE
45 model incorporates key features, such as the thermal resistance at the steel tube and concrete
46 interface, the variation of the Poisson’s ratio of concrete at elevated temperatures and the variation of
47 the bi-axial compressive strength to uniaxial compressive strength ratio of concrete at elevated
48 temperatures. The FE model was validated against the fire test results presented herein on the
49 STCRC columns, as well as existing tests on CFST columns (Lie and Chabot 1992; Chabot and Lie
50 1992; Espinos et al. 2015). Following validation of the model, the influence of key parameters on the
51 cross-sectional temperature development are identified, and simplified methods are proposed for
52 predicting the temperatures of the outer steel tube, inner concrete and reinforcing bars. A design
53 method for evaluating the load-bearing capacity of STCRC columns exposed to fire is then presented,
54 which can be employed to predict the fire resistance of STCRC columns.

55 **Experimental program**

56 **General information**

57 In the experimental program, five full-scale specimens, including four circular steel tube confined
58 reinforced concrete (STCRC) columns and one concrete-filled steel tubular (CFST) column, were
59 tested to failure under the ISO 834 standard fire conditions (1999). Plain concrete-filled steel tubular
60 columns are usually used in practice in China and hence a plain CFST column was employed herein
61 for comparison purposes. The influence of three parameters on the fire behavior of these columns
62 was investigated: load ratio ($n=0.25, 0.33, 0.42$), frequency of breaks to the longitudinal continuity
63 of the steel tube (case (a) and case (b), which are defined below) and type of specimen (STCRC or
64 CFST). Table 1 summarizes the details of the specimens, where D is the cross-sectional diameter, t_s
65 is the wall thickness of the steel tube, L is the length of the specimens, α_s is the steel tube to concrete
66 area ratio, α_b is the reinforcement ratio, N_f is the applied axial load and n is the load ratio ($n=N_f/N_u$,
67 where N_u is the ultimate capacity of the specimen before fire exposure determined by FE analysis).

68 In order to prevent the vertical load from being directly applied to the steel tube of the STCRC
69 columns, two strips were cut from the steel tubes, introducing 30 mm wide gaps (as shown in Fig.1).
70 However, axial load may be transferred to the steel tube via the friction and bond between the steel
71 tube and the concrete in practice. To investigate the degree and influence of the axial load transferred
72 to the steel tube, the frequency of breaks to the longitudinal continuity of the steel tube was varied,
73 considering two cases: case (a) and case (b). For case (a), the central fire-exposed portion of the steel
74 tube was continuous, as shown in Fig.1(a). For case (b), the exposed portion of the steel tube was
75 divided into three equal parts (see Fig.1(b)), thereby breaking the longitudinal continuity more
76 frequently and decreasing the possible vertical load borne by the steel tube. The steel tube in the
77 CFST column was continuous along the whole length and subjected to axial loading as well. For
78 each column, the fire-exposed length was approximately 3000 mm. All specimens had an outer
79 diameter of 300 mm and a length of 3810 mm. The steel tube to concrete area ratio and the
80 reinforcement ratio of the STCRC columns were about 3.38% and 3.68%, respectively. The total
81 steel ratio (i.e. tube plus reinforcement) of the STCRC columns was 7.06%, which is similar to that
82 of the CFST column (7.58%).

83 **Material properties**

84 The properties of the steel tube were obtained by means of tensile coupon testing, conducted in
85 accordance with ISO 6892-1 (2009). The measured yield strength f_{sy} , ultimate tensile strength f_{su} ,
86 elastic modulus E_s and Poisson's ratio ν_s of the structural steel at ambient temperature are presented
87 in Table 2. Hot-rolled ribbed bars and hot-rolled plain bars were used for the longitudinal reinforcing
88 bars and stirrups, respectively. The properties of the reinforcing bars obtained from tensile coupon
89 tests are given in Table 3, where d_b is the diameter, f_{by} is the yield strength, f_{bu} is the ultimate tensile
90 strength and E_b is the elastic modulus.

91 Ready-mixed concrete was used to fill the steel tubes. Concrete cubes ($150\text{ mm} \times 150\text{ mm} \times 150\text{ mm}$)
92 and concrete prisms ($150\text{ mm} \times 150\text{ mm} \times 300\text{ mm}$) were cast during the preparation of the
93 specimens and wrapped in aluminum foil to simulate the closed environment that the concrete
94 experiences in the steel tubes. The cube and prism specimens were cured, together with the columns,
95 in the testing laboratory. The concrete cube strength and elastic modulus were tested at 28 days after
96 pouring and on the test day of the first specimen; the results are given in Table 4.

97 **Fabrication of specimens**

98 The steel tubes were produced by seam welding cold-formed steel sheets, and the ends of the
99 members were machined flat and parallel. The reinforcing cages were fabricated by tying eight
100 longitudinal reinforcing bars with stirrups at intervals of 200 mm, and were placed inside the steel
101 tubes. The concrete cover from the edge of the concrete to the perimeter of the reinforcing bars was
102 around 25 mm. Two 20 mm thick end plates, each with four stiffening ribs, were welded to the ends
103 of the steel tube to ensure the end sections remained plane during testing. The geometrical center of
104 the steel tube coincided with that of the end plates to attain concentric loading conditions.

105 For the CFST column, two semi-circular holes, with a diameter of 20 mm, were drilled at each end
106 of the steel tube to provide vent holes for water vapor produced during the test. The water vapor in
107 the STCRC columns could be released via the gaps, and thus vent holes were not provided.

108 Holes of 30 mm diameter were drilled at the mid-height and near the ends of the specimens for the
109 installation of thermocouples, as shown in Fig.1. Type K (chromel/alumel) thermocouples, with a
110 diameter of 1 mm, were installed to measure the temperatures at different locations across the section.

111 The resulting gaps were filled with high-strength non-shrinkage grout. The layout of the
112 thermocouples is depicted in Fig.2.

113 **Test setup and procedure**

114 The specimens were tested in a custom-built furnace for testing structural members under combined
115 thermal and structural loading conditions, at Harbin Institute of Technology. The loading capacity is
116 300 tonnes. The furnace has a 4.9 m × 2.6 m floor area and a height of 4.05 m. The interior surfaces
117 of the furnace chamber are lined with ceramic fiber materials that efficiently transfer heat to the test
118 specimens. Eight platinum-rhodium thermocouples were installed at different locations within the
119 furnace to measure the air temperature. The furnace temperature can be automatically controlled to
120 follow programed air temperature-time curves. A special camera, which can endure high
121 temperatures by utilizing both air cooling and water cooling, was mounted at the mid-height of the
122 chamber to capture the response of the specimens during testing. The specimens were fixed at the
123 bottom end and pinned at the top end. Concentric load was applied to the specimens around 30 mins
124 prior to the ignition of the fire and then held constant. The column is 3770 mm in length and only
125 3000 mm was exposed to fire directly. The portion of the column length outside the furnace was 210
126 mm. A general view of test setup is shown in Fig.3.

127 The ambient temperatures ranged from 20 °C to 24 °C before the tests. The furnace temperatures
128 were increased following the ISO 834 standard fire curve. The furnace and specimen temperatures
129 and the axial displacement versus time curves of the specimens were monitored and recorded during
130 the tests. The failure criteria specified in ISO 834 (1999) were adopted in this study – when the axial
131 deformation reached 0.01*L* mm or the rate of axial deformation reached 0.003*L* mm/min, where *L* is
132 the length of the specimens in mm, the specimens were considered to have failed and the tests were
133 terminated.

134 **Test results**

135 The deformed STCRC and CFST columns after testing are presented in Fig.4. The STCRC columns
136 failed by global buckling, accompanied by local buckling (Fig.4(f)). The CFST column failed with
137 no significant lateral deformation, but clear evidence of local buckling of the steel tube (Fig.4(e) and
138 (g)). There is no obvious lateral deformation for specimen STCRC-1 because the supply of gas was

139 exhausted when the column was close to failure and the lateral deformation recovered as the axial
140 load was removed. A typical specimen after removal of the steel tube is shown in Fig.5. Despite
141 some crushing of the concrete, the inner concrete remained essentially intact, which may be
142 attributed to the presence of the outer steel tube.

143 The temperatures in the furnace were uniform and followed the ISO 834 standard fire curve
144 accurately, as shown in Fig.6, in which T1 to T8 are the measured temperatures from the eight
145 thermocouples in the furnace during the testing of specimen STCRC-1. The average furnace
146 temperatures of all tests agreed well with the ISO 834 standard fire curve, and the deviation of the
147 average furnace temperature from the ISO 834 standard temperature curve was within the limits
148 specified in ISO 834 (1999) (see Fig.7), confirming the reproducibility and robustness of the furnace.

149 The temperatures of corresponding thermocouples at different heights, e.g. thermocouple 2 and 8, are
150 very close, as depicted in Fig.8, confirming the uniformity of temperature along the longitudinal
151 direction of the specimens and further confirming the uniformity of temperature in the furnace.

152 Thermocouple 6 failed during testing, after which no results were available. There are some
153 discrepancies of temperature in the steel tube (Fig.8(a)), which may be attributed to the sensitivity of
154 the thermocouples to flames or the influence of the insulation materials positioned to ensure
155 thermocouples will not be subjected to flame directly.

156 The measured temperatures of the specimens through the cross-sections during the tests are
157 presented in Fig.9. The rate of increase in temperature of the steel tube decreases with increasing
158 time, similar to the ISO-834 standard fire curve. Take specimen STCRC-1 for example – the
159 temperature of the steel tube reached 320 °C, 471 °C, 688 °C and 880 °C after heating times of 10
160 min, 20 min, 40 min and 80 min, respectively. The deeper the location of the thermocouples within
161 the concrete, the lower the temperature that was achieved, and the longer it took for the onset of
162 rising temperatures. Considering specimen STCRC-1, the temperatures began to rise for the
163 thermocouples with a distance of 50 mm, 100 mm and 150 mm from the steel tube, from about 5 min,
164 15 min and 20 min after ignition, respectively. This delay in temperature rise is due to the high
165 thermal capacity of the concrete and the insulation effect of the surrounding material. Owing to
166 moisture evaporation and migration, the temperature of the concrete remained essentially constant

for some time at around 100 °C, after which the temperature continued to increase. The measured axial displacement versus time curves are presented in Fig.10, in which positive and negative values correspond to expansion and contraction, respectively. The axial displacement velocity versus time curves are also depicted in Fig.10. The axial displacement versus time curves may be seen to exhibit three stages: (i) expansion; (ii) stable shortening and (iii) rapid shortening. These three stages were also observed in fire tests on square and rectangular CFST columns (Yang et al. 2013a; 2013b). The fire resistances of the columns were determined according to the criteria specified in ISO 834 (1999) and are indicated in Fig.10. Specimen STCRC-1 was not tested to failure, but it was close to failure by the end of the test, as can be observed from the displacement versus time curve in Fig.10(a). Therefore, the time at the end of the test was taken as the approximate fire resistance of specimen STCRC-1 in subsequent analysis.

Discussion of results

The influence of load ratio on the axial displacement versus time curves and fire resistance of the columns is depicted in Fig.11 (a). The expansion stage may be seen to reduce with increasing load ratio, with even no expansion arising in the case of high load levels. The maximum expansion was 1.8 mm and 0.6 mm for specimens with a load ratio of 0.25 and 0.33, respectively, while the corresponding duration of expansion was 88 min and 47 min, respectively. However, only shortening occurred for the specimen with a load ratio of 0.42. The expansion depends predominantly on the temperature and the length of the specimen, whereas the shortening is dominated by the load ratio and the temperature, hence the observed decrease in the expansion stage with increasing load ratio. The load ratio significantly influenced the fire resistance of the STCRC columns. In this study, the fire resistances were 116.5 min, 82.5 min and 50 min for the specimens with load ratios of 0.25, 0.33 and 0.42, respectively, decreasing by 29.2% and 57.1% for the specimens with the load ratios of 0.33 and 0.42 respectively relative to the specimen with the load ratio of 0.25.

The frequency of the breaks to the longitudinal continuity of the members had limited influence on the deformation and fire resistance of the STCRC columns, as shown in Fig.11 (b). This can be explained as follows: (1) The high temperature deteriorates the bond stress between the steel tube and concrete, and hence the axial load transferred to the steel tube through the bond stress decreases.

(2) The temperature of the steel tube rises more rapidly than that of concrete, causing degradation of the mechanical properties and lateral thermal expansion of the steel tube, rendering the confinement afforded by the steel tube to the concrete relatively insignificant for columns in fire. In spite of this, the steel tube is effective in preventing spalling and retaining the integrity of the concrete.

The development of the deformation of the STCRC columns generally follows that of the CFST column shown in Fig.11 (c). However, the STCRC columns possessed much higher fire resistance than the CFST column with the same load level ($n=0.42$), achieving over double the fire exposure time at failure. The greatly improved fire performance of the STCRC columns is primarily due to the thermal protection afforded to the embedded steel, which plays an important load-bearing role with increasing temperature, in contrast to the outer steel tube in CFST columns that is directly exposed to the fire.

Although not explicitly investigated in the present study, the fire resistance of STCRC columns relative to reinforced concrete (RC) columns has been examined in previous research (Liu et al., 2017). It was found that STCRC columns have lower fire resistance than RC columns, with greater differences observed at higher load levels. The lower fire performance of the STCRC columns relates principally to the high load-bearing capacities at room temperature that benefit from the confinement afforded to the concrete being eliminated by the rapid deteriorations of the material properties and hence loss of confinement ability of the outer tube in fire.

Development of the FE model

A finite element (FE) model was developed employing a sequentially coupled thermal-stress analysis, whereby a heat transfer analysis was first conducted, and then the thermal results were imported into a subsequent stress analysis.

Heat transfer analysis

The steel tube, reinforcing bars and concrete were modelled with 4-node shell elements (DS4), 2-node truss elements (DC1D2) and 8-node continuum solid elements (DC3D8), respectively. The ambient temperature was set at 20 °C and the temperature was increased following the ISO 834 (1999) standard fire curve. Recognizing the symmetry of the geometry and boundary conditions, a quarter of the column was modelled, as shown in Fig.12.

223 In the analysis, the heat is transferred from the fire to the outer steel tube via convection and
224 radiation, and then into the columns by means of conduction. The convective heat transfer coefficient
225 and the resultant emissivity were taken as $25 \text{ W}/(\text{m}^2\text{K})$ and 0.5, respectively (Ding and Wang 2008;
226 Yang et al. 2013a, 2013b; Liu et al. 2014). The thermal resistance, arising from air gaps between the
227 steel tube and the concrete, was taken into account with a constant value of $0.01 (\text{m}^2\text{K})/\text{W}$ (Ding and
228 Wang 2008; Lv et al. 2011), which was found to provide accurate predictions (Liu et al.2014; Yang et
229 al.2015).

230 The density, thermal conductivity and specific heat of the materials are required for the heat transfer
231 analysis. The thermal models recommended by Lie et al. (1994), which have been successfully used
232 by Hong and Varma (2009) and Yang et al. (2013a; 2013b) to simulate CFST columns, were
233 employed in this study, since the cross-section of STCRC columns resembles that of CFST columns.
234 Han and his research group conducted numerous studies on CFST structures exposed to fire using
235 these thermal models, including circular, square and rectangular CFST columns during and after fire
236 exposure (Han et al. 2003a; 2003b; 2003c; 2005b; Yang et al. 2008; Song et al. 2010a), beam to
237 column connections (Tan et al. 2012; Song et al. 2010b) and frames (Han et al. 2012) exposed to fire,
238 confirming the robustness of the thermal models. The thermal models provided in Eurocode 4 have
239 also been successfully employed for the simulation of CFST columns by many researchers
240 (Schaumann et al. 2009; Ding and Wang 2008; Espinos et al. 2010; Ibañez et al. 2013; Imani et al.
241 2015; Neuenschwander et al. 2017a), with good predictions obtained. These models would also be
242 expected to be suitable for the simulation of STCRC columns exposed to fire. Moisture evaporation
243 and migration affect the temperature development of the concrete; reasonable temperature
244 predictions can generally be obtained by taking the moisture evaporation into account but ignoring
245 the moisture migration. Different values of moisture content have been adopted by different
246 researchers, ranging from about 4% to 10% by weight, as summarised by Tao and Ghannam (2012).
247 Tao and Ghannam (2012) proposed an equation for calculating the moisture content of the concrete
248 in CFST columns based on the mix proportions; a value of 5% by weight was recommended if the
249 mix proportions of the concrete are not detailed. The moisture content of the concrete in the STCRC
250 columns was assumed to be 5% by weight herein.

251 **Stress analysis**

252 **Elements, interactions and initial imperfections**

253 Following the heat transfer analysis, nonlinear stress analysis was conducted, allowing the behavior
254 of STCRC columns to be investigated under combined thermal and structural loading. In order to
255 ensure that the thermal results could be read into the stress analysis model efficiently and accurately,
256 the same finite element mesh was employed for both models, but the heat transfer elements were
257 replaced by stress elements. For the stress analysis, the elements of the steel tube, the reinforcing
258 bars and the concrete were changed to linear reduced-integration 4-node shell elements (S4R),
259 2-node truss elements (T3D2) and linear reduced-integration 8-node solid elements (C3D8R),
260 respectively.

261 The interactive behavior between the steel tube and the concrete was simulated with the contact
262 interaction algorithm in ABAQUS. The surfaces of the steel tube and the concrete were defined to be
263 a contact pair, with the inner surface of the steel tube as the slave surface and the outer surface of the
264 concrete as the master surface. The interface behavior in the normal direction was defined to be
265 “hard” contact, which means the contact pressure is fully transmitted when the surfaces are touching.
266 In the tangential direction, Coulomb friction behavior with a coefficient of friction of 0.3 was defined.
267 The embedded region constraint was used to embed the reinforcing bars within the concrete.

268 Initial imperfections were incorporated into the models in the form of the first elastic buckling mode
269 shape with an amplitude of $L/1000$, where L is the length of the column, this approach has been
270 widely used in previous studies (e.g., Espinos et al. 2011). For the STCRC columns, load was only
271 applied to the concrete core, while for the CFST column simulated for the model validation, load was
272 applied to the whole cross-section.

273 **Steel properties**

274 Steel under load at elevated temperatures experiences mechanical strain, thermal strain and creep
275 strain. The mechanical stress-strain relationship provided in EC3 (2005), which implicitly
276 incorporates the effects of creep, was found by Espinos et al. (2010) and Wang and Young (2013) to
277 yield better predictions of the fire behavior of CFST columns than the models of Lie (1994) and Poh
278 (2001). Similar findings were presented by Kodur et al. (2010), where the EC3 (2005) material

model was recommended in preference to the models of ASCE (1992) and Poh (2001). It should be noted that ASCE model (1992) is the same as that of Lie (1994). The EC3 (2005) model was also employed in the present study, and shown to yield accurate results.

Espinos et al. (2010) found that the EC3 (2005) thermal expansion model produces more accurate predictions than the constant thermal expansion coefficient value of 1.2×10^{-5} , proposed by Hong and Varma (2009), when studying the fire performance of CFST columns. Lu et al. (2009) and Yang et al. (2013a; 2013b) also gained good predictions with the thermal expansion model recommended by Lie (1994).

The relative thermal elongation $\Delta l/l$ of steel recommended in EC3 (2005) is as follows:

$$\Delta l/l = \begin{cases} 1.2 \times 10^{-5} T + 0.4 \times 10^{-8} T^2 - 2.416 \times 10^{-4} & 20^\circ\text{C} \leq T < 750^\circ\text{C} \\ 1.1 \times 10^{-2} & 750^\circ\text{C} \leq T \leq 860^\circ\text{C} \\ 2 \times 10^{-5} T - 6.2 \times 10^{-3} & 860^\circ\text{C} < T \leq 1200^\circ\text{C} \end{cases} \quad (1)$$

where T is the temperature in $^\circ\text{C}$. The corresponding thermal expansion coefficient α is:

$$\alpha = \begin{cases} 1.2 \times 10^{-5} + 0.8 \times 10^{-8} (T - 20) & 20^\circ\text{C} \leq T < 750^\circ\text{C} \\ 0 & 750^\circ\text{C} \leq T \leq 860^\circ\text{C} \\ 2 \times 10^{-5} & 860^\circ\text{C} < T \leq 1200^\circ\text{C} \end{cases} \quad (2)$$

The thermal expansion coefficient proposed by Lie (1994) is as follows:

$$\alpha = \begin{cases} (0.004T + 12) \times 10^{-6} & T < 1000^\circ\text{C} \\ 16 \times 10^{-6} & T \geq 1000^\circ\text{C} \end{cases} \quad (3)$$

Comparisons of the thermal expansion coefficient and the relative thermal elongation between the EC3 model and the Lie model are shown in Fig.13. Despite discrepancies of the thermal expansion coefficient, the relative thermal elongations are very close for these two models. However, the EC3 model accounts for the phase change that occurs in steel between about 750°C and 860°C by assuming a constant thermal elongation in this temperature range, which agrees well with test results on steel at elevated temperatures (Cooke 1988; Outinen 2007). Therefore, the EC3 model was adopted in this study.

Concrete properties

Strains in concrete under load at elevated temperatures include four components: the mechanical strain, the thermal strain, the creep strain and the transient strain. The stress-strain relationship

recommended by Lie (1994), which is also the ASCE model (1992), was adopted herein. It implicitly incorporates creep strain and has been found to yield better predictions of the behavior of CFST columns than the EC2 model (2004) and the Li and Purkiss model (2005), by Espinos et al. (2010) and Hong and Varma (2009). The thermal expansion coefficient of concrete was taken as a constant value of 6×10^{-6} , as recommended by Espinos et al. (2010) and Hong and Varma (2009). The transient strain model proposed by Anderberg and Thelandersson (1976), which has been widely used for the structural analysis of members exposed to fire (Sadaoui and Khennane 2009; Capua and Mari 2007; Kodur and Dwaikat 2008), was employed in this study and is given as follows:

When $T \leq 550^\circ\text{C}$

$$\varepsilon_{\text{ctr}} = k \left(\frac{\sigma}{f'_c} \right) \varepsilon_{\text{cth}} \quad (4)$$

When $T > 550^\circ\text{C}$

$$\frac{\partial \varepsilon_{\text{ctr}}}{\partial T} = -0.0001 \left(\frac{\sigma}{f'_c} \right) \quad (5)$$

where ε_{cth} is the thermal strain, σ is the stress, f'_c is the compressive cylinder strength of the concrete and k is a factor between 1.8 and 2.35; an intermediate value of 2.0 was used herein. The transient strain was added to the mechanical strain and the stress-total strain relationship was then input into ABAQUS (Li and Purkiss 2005).

For the structural steel and reinforcing bars, the classical metal plasticity model with von Mises yielding and the associated plastic flow rule was used, with the stress-strain relationship for both following that set out in EC3 (2005). The reduction factors for the strength and elastic modulus of the structural steel and reinforcing bars at elevated temperatures were taken from EC3 (2005) and EC2 (2004), respectively. The elastic modulus of both the structural steel and reinforcing bars at ambient temperature was defined to be $2.1 \times 10^5 \text{ N/mm}^2$, according to EC3 (2005). The influence of temperature on the Poisson's ratio of steel is small, hence the Poisson's ratio was set at a constant value of 0.3 for both the structural steel and reinforcing bars.

The concrete damage plasticity model in ABAQUS was used to represent the concrete material, which requires the definition of the behavior in both compression and tension. The concrete models

proposed by Lie (1994) and Hong and Varma (2009) were used to define the compressive and tensile behavior of the concrete, respectively. Tests on concrete material after exposure to elevated temperatures conducted by Marechal (1972) and Luccioni et al. (2003) indicate that the Poisson's ratio of concrete tends to decrease with increasing temperature. Based on these post-elevated temperature test results, Gernay et al. (2013) proposed an equation reflecting the relationship between Poisson's ratio and temperature. The suitability of the equation was confirmed up to a temperature of 300 °C by the first elevated temperature Poisson's ratio test conducted by Bahr et al. (2013). However, there are some discrepancies between the equation of Gernay et al. and the test results of Bahr et al. (2013) for temperatures between 300 °C and 500 °C. To date, there are no available test data for the Poisson's ratio of concrete at temperatures exceeding 500 °C. Clearly further test data are required on this topic, but for the purposes of the present study, the recommendations of Gernay et al. (2013) are adopted, which are as follows:

$$\nu_{cT} = \begin{cases} \nu_c \left(0.2 + 0.8 \frac{500 - T}{480} \right) & T \leq 500^\circ\text{C} \\ 0.2\nu_c & T > 500^\circ\text{C} \end{cases} \quad (5)$$

where ν_{cT} and ν_c are the Poisson's ratio of concrete at elevated temperature and ambient temperature, respectively. ν_c is taken as 0.2.

Ehm and Schneider (1985) found that bi-axial compressive strength displays a relative increase with increasing temperature due to the restraining of thermal and load induced micro-cracking. These tests were conducted at temperatures up to 600 °C. Based on these test results, Gernay et al. (2013) proposed an equation reflecting relationship between the ratio of the bi-axial compressive strength $f_{bi,cT}$ to the uniaxial compressive strength f_{cT} and temperature, as given by:

$$f_{bi,cT} / f_{cT} = \begin{cases} 1.16 & T \leq 350^\circ\text{C} \\ 1.16 \left(1 + 0.6 \frac{T - 350}{400} \right) & 350^\circ\text{C} < T \leq 750^\circ\text{C} \\ 1.86 & T > 750^\circ\text{C} \end{cases} \quad (6)$$

This relationship was adopted in this study and was incorporated into ABAQUS.

The damage part of the concrete damage plasticity model was neglected in all studies on fire performance of structural members and good predictions were obtained. This can be explained by

two reasons: 1) there may be no obvious process of unloading and reloading, 2) there is no available model to represent concrete damage at elevated temperatures under cyclic loading. For STCRC columns, axial load is mainly sustained by inner reinforced concrete column, and there is no process of unloading and reloading. Therefore damage part of the concrete damage plasticity model also was not considered in this simulation. It should be mentioned that recently Neuenschwander et al. (2016; 2017b; 2018) validated the suitability of concrete damage plastic model for concrete at elevated temperatures for the first time experimentally and studied the damage evolution of elastic modulus at elevated temperatures. It will be very beneficial for simulation of composite structures exposed fire, especially for indeterminate structures.

Validation of FE model and parametric studies

Validation of FE model

The temperature history (temperature-time response) in the tested circular STCRC columns was simulated using the developed FE model, as shown in Fig.14. The cross-section of STCRC columns resembles that of CFST columns; the thermal analysis model was therefore also validated against tests on CFST columns. Typical comparisons of the predicted and test temperatures of plain concrete-filled steel tubular columns (Lie and Chabot 1992) and reinforced concrete-filled steel tubular columns (Chabot and Lie 1992) are illustrated in Fig.15 and Fig.16, respectively, where d is the distance of the thermocouple to the outer surface of the concrete. The geometrical details of these CFST columns are provided in Table 5; note that the furnace temperature for these columns was increased following the ASTM-E119 fire curve (1990). The predicted temperatures may be seen to generally accord well with the test temperatures, for both the STCRC columns and CFST columns. There are some discrepancies between the predicted and test temperatures in the concrete when the temperature is around 100 °C, due to the complexities in simulating of the effect of moisture. However, these discrepancies have an insignificant influence on the prediction of fire resistance since there is no reduction in the concrete strength at that temperature level.

The deformation-time responses of the STCRC columns in fire were also predicted using the developed FE model, as depicted in Fig.17. Fire tests on STCRC columns are limited, and since the composition of CFST columns is similar to that of STCRC columns, other than that the outer steel

tube in CFST columns is subjected to direct axial loading, test results on CFST columns were also employed to validate the FE model. The plain concrete-filled steel tubular columns tested by Lie and Chabot (1992) and Han et al. (2003a) and the reinforced concrete-filled steel tubular columns tested by Chabot and Lie (1992) and Espinos et al. (2015), were simulated using the developed FE model. Typical comparisons between the FE and test results are presented in Fig.18 and Fig.19. The details of these columns are given in Table 6 and Table 7, respectively, in which D is the outer diameter of the steel tube, t_s is the wall thickness of the steel tube, L is the length of the specimen, e is the load eccentricity, f_c' is the concrete cylinder strength, f_y is the yield strength of the structural steel, N_f is the applied axial load and a is the thickness of the fire protection.

The predicted and test fire resistances of the STCRC columns, plain concrete-filled steel tubular columns and reinforced concrete-filled steel tubular columns are shown in Fig.20. The mean and standard deviation of the ratio of the predicted-to-test fire resistance are 1.072 and 0.237, respectively. The discrepancies may be attributed to: (i) the complexity of fire resistance testing and possible inaccuracies in the measurements and (ii) the assumptions made in the FE model, for example, the stress-strain relationships of the steel and concrete were not taken directly from tests. Despite these potential uncertainties, the predicted fire resistances generally agree well with the test results.

Parametric studies

After validation of the FE model, parametric studies were carried out to extend ranges of studied parameters, including load ratio (n), cross-section dimension (D), slenderness ratio ($\lambda = L/i$, where L is column length and i is the radius of gyration), concrete strength (f_c'), yield strength of steel (f_y), yield strength of reinforcement (f_b), steel tube to concrete area ratio (α_s) and reinforcement ratio (α_b). Ranges of these parameters are as follows: $n=0.3 - 0.7$, $D=200 \text{ mm} - 1500 \text{ mm}$, $f_c'=24 \text{ N/mm}^2 - 50 \text{ N/mm}^2$, $f_y=235 \text{ N/mm}^2 - 420 \text{ N/mm}^2$, $f_b=335 \text{ N/mm}^2 - 500 \text{ N/mm}^2$, $\alpha_s=2\% - 4\%$, $\alpha_b=2\% - 5\%$. The default condition is taken as: $D=600 \text{ mm}$, $f_c'=40 \text{ N/mm}^2$, $f_y=345 \text{ N/mm}^2$, $f_b=335 \text{ N/mm}^2$, $\alpha_s=3\%$, $\alpha_b=4\%$.

Influences of these parameters on fire resistance are presented in Fig.21. Fire resistance decreases significantly with the increase of load ratio, as shown in Fig.21 (a). Take the column with a diameter

of 600 mm for example, its fire resistance decreases from 157.7 min to 30.3 min when load ratio increases from 0.4 to 0.6. Higher load ratio means higher load is applied, and then the load-bearing capacity declines to the level of applied load in a shorter time.

Fire resistance increases with increase of cross-section dimension, concrete strength, yield strength of reinforcement and reinforcement ratio, while it decreases with increase of slenderness ratio, yield strength of steel and steel tube to concrete area ratio (Fig.21 (b)-(h)). Influences of these parameters on fire resistance of circular STCRC columns resemble to that of CFST columns. When the load ratio is within 0.5, influences of these parameters on fire resistance are obvious, whereas influences of these parameters are limited while the load ratio exceeds 0.5. Han et al. (2013a) and Yang et al. (2013b) found steel tube to concrete area ratio and strengths of concrete and steel have limited influence on the fire resistance of CFST columns, since load ratio exceeds 0.5 in their studies.

Simplified design methods

All above parameters have some influence on the fire resistance of circular STCRC columns, especially for columns with a load ratio of less than 0.5. It is infeasible to propose equations for predicting fire resistance directly from parametric study results by regression analysis. Since the load-bearing capacity of columns degrades with increasing temperature, once the load-bearing capacity declines to the level of applied load, the column fails and the corresponding time is the fire resistance of the column. Simplified design methods for predicting the temperature and load-bearing capacity of STCRC columns are sought in this section, which can be used to determine fire resistance.

Design method for predicting temperatures

The cross-section of STCRC columns resembles that of CFST columns. The Chinese code GB50936 (2014) and the Japanese code AIJ (2008) provide simplified methods for calculating the temperature of the steel tube in CFST columns exposed to the standard fire. The equation given in the Chinese code GB50936 (2014) is as follows:

$$T_s = A \left[1 - \frac{1}{1 + \left(\frac{t}{B} \right)^C} \right] + 20 \quad (7)$$

435 where $A=1200$, $B=0.337+8.5t_s$ and $C=0.996+14.0t_s$, t_s is the wall thickness of the steel tube in meters,
436 and t is the heating time in hours.

437 The equation provided by the Japanese code AIJ (2008) is as follows:

$$438 \quad T_s = 1080 - 450 \exp(-0.8t) - 630 \exp(-3t) + 20 \quad (8)$$

439 where t is the heating time in hours.

440 The predicted temperatures using these equations and those obtained from the FE models are
441 compared in Fig.22, revealing good agreement. Therefore the temperature of the steel tube in
442 STCRC columns may be approximated using either Eq.(7) or Eq.(8).

443 Similar to reinforced concrete columns, the temperatures of the reinforcing bars in STCRC columns
444 can be represented by the temperature of the concrete at the location of the center of the bar section.
445 The Japanese code AIJ (2008) provides a method for calculating the concrete temperature, given as
446 follows:

$$447 \quad T_c = (T_s - 20) \exp \left(\frac{10(2x/D_c - 1)}{(t/D_c^2)^{3/4}} \right) + 20 \quad (9)$$

448 where x is the distance to the center of the concrete in centimeters, D_c is the diameter of the core
449 concrete in centimeters, and t is the heating time in seconds. In Eq.(9), when x equals $0.5D_c$, T_c
450 equals T_s , which means that the thermal resistance at the interface between the steel tube and the
451 concrete is not accounted for in this equation. The influence of the thermal resistance at the interface
452 on the temperature of the concrete was investigated with the FE model, as shown in Fig.23. The
453 thermal resistance may be seen to have a significant influence on the temperature near the concrete
454 surface (i.e. $d=0$). However, the influence degrades quickly as the distance to the concrete surface d
455 increases. The concrete cover to the reinforcing bars usually exceeds 20 mm, so the distance of the
456 center of the bar section to the concrete surface would typically be more than 30 mm. Therefore the
457 thermal resistance has little influence on the temperatures of the reinforcing bars, and Eq.(9) may be
458 used to approximate the temperatures of the reinforcing bars. When the units used in Eq.(9) are
459 replaced by SI units, the equation becomes:

$$T_b = (T_s - 20) \exp \left(-43.03 \frac{d \sqrt{D_c}}{t^{3/4}} \right) + 20 \quad (10)$$

where d is the distance to the concrete surface in meters, D_c is the diameter of the core concrete in meters, and t is the heating time in hours.

The temperature of the concrete is non-uniform across the section due to the high thermal capacity and low thermal conductivity. The concrete cross-section can be subdivided into a number of layers, and the temperature of each layer can be calculated from Eq.(9). However, this method is often not practical for use in engineering design. An alternative approach is to determine an equivalent uniform temperature for the whole cross-section, which yields the same cross-section capacity as the real non-uniform temperature distribution. The equivalent compressive strength of concrete can be determined as follows:

$$f_{c,eq}(T) = \frac{\sum_{i=1}^n f_{ci}(T) A_{ci}}{A_c} \quad (11)$$

where i is the element number, $f_{ci}(T)$ is the compressive strength of the element i , A_{ci} is the area of element i , n is the total number of elements across the section and A_c is the area of the full concrete core.

The reduction factor for the compressive strength of concrete at elevated temperatures is as follows:

$$k_{cT} = \frac{f_{c,eq}(T)}{f_c} \quad (12)$$

According to the compressive strength versus temperature relationship proposed by Lie (1994), the equivalent temperature of concrete is:

$$T_{c,eq} = \frac{(2.011 - k_{cT}) \times 10^3}{2.353} + 20 \quad (13)$$

When $k_{cT}=1.0$, $T_{c,eq}=449.7^\circ\text{C}$, and when $k_{cT}=0$, $T_{c,eq}=874.7^\circ\text{C}$.

A total of 91 parametric FE results for STCRC columns with varying diameters (from 200 mm to 1500 mm) and subjected to a range of heating times (0 to 3 hours) were obtained to assess the temperature development in the cross-sections, and hence, by means of Eqs (12) and (13), the reduction factors for the compressive strength of concrete at elevated temperatures. A predictive

equation for the reduction factor k_{cT} was then established based on the parametric results, which is given as follows:

$$k_{cT} = 1 - \left(\frac{0.045}{D_c} - 0.016 \right) t \quad (14)$$

where D_c is the diameter of the concrete core in meters and t is the heating time in hours.

Comparisons of the predicted reduction factor from Eq.(14) and the FE model are depicted in Fig.24, which shows good agreement. Based on this reduction factor, the following simplified equation is proposed for calculating the equivalent temperature of concrete:

$$T_{c,eq} = 449.7 + 425 \left(\frac{0.045}{D_c} - 0.016 \right) t \quad (15)$$

Design method for predicting load-bearing capacity

Liu and Zhou (2010) proposed a design method for predicting the cross-sectional capacity of STCRC columns at ambient temperature, which is given as follows:

$$N_0 = f_{cc} A_c + f_b A_b \quad (16)$$

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 (Mander et al. 1988):

$$f_{cc} = \left(-1.254 + 2.254 \sqrt{1 + 7.94 \frac{f_r}{f_c'}} - 2 \frac{f_r}{f_c'} \right) f_c' \quad (17)$$

where f_c' is the concrete cylinder strength and f_r is the confining stress imposed by the steel tube on the concrete, which can be calculated as follows:

$$f_r = \frac{2t_s f_y}{D - 2t_s} \quad (18)$$

where f_y is the yield strength of the steel tube, and D and t_s are the cross-sectional diameter and thickness of the steel tube, respectively.

Consistent with the above method for STCRC columns at ambient temperature, a design method is proposed herein for predicting the cross-sectional capacity of STCRC columns subjected to the ISO 834 standard fire.

$$N_0(T) = f_{cc}(T)A_c + f_b(T)A_b \quad (19)$$

where

$$f_{cc}(T) = \left(-1.254 + 2.254 \sqrt{1 + 7.94 \frac{f_r(T)}{f'_{c,eq}(T)}} - 2 \frac{f_r(T)}{f'_{c,eq}(T)} \right) f'_{c,eq}(T) \quad (20)$$

$$f_r(T) = \frac{2t_s f_y(T)}{D - 2t_s} = \frac{2t_s k_{yT} f_y}{D - 2t_s} \quad (21)$$

$$\text{and } f_b(T) = k_{bT} f_b \quad (22)$$

where $f_{c,eq}(T)$ can be obtained by substituting the equivalent temperature $T_{c,eq}$ from Eq. (16) into the model proposed by Lie (1994):

$$f'_{c,eq}(T) = \begin{cases} f'_c & 0^\circ\text{C} < T_{c,eq} < 450^\circ\text{C} \\ \left(2.011 - 2.353 \left(\frac{T_{c,eq} - 20}{1000} \right) \right) \beta f'_c & 450^\circ\text{C} \leq T_{c,eq} \leq 874^\circ\text{C} \\ 0 & T_{c,eq} > 874^\circ\text{C} \end{cases} \quad (22)$$

where β is a factor proposed herein to account for the effect of the differential stresses within the cross-section under the same strain, due to the large thermal gradient within sections under the ISO 834 standard fire; the factor β can be calculated as follows based on the results of the thermal parametric study.

$$\beta = \frac{1}{1 + (0.005t + 0.2) \sqrt{\frac{t}{D_c}}} \quad (23)$$

For columns at ambient temperature or having a uniform cross-section temperature (e.g. 200 °C), the factor β equals 1.0 since there is no thermal gradient. Comparisons of the predicted load-bearing capacities using the simplified design method with those obtained from the FE models are shown in Fig.25 (a) for the case of uniform temperature and Fig.25 (b) for the case of exposure to the ISO 834 standard fire. It may be seen that the thermal gradients in the concrete have a significant influence on the cross-sectional capacity of circular STCRC columns exposed to the ISO 834 standard fire and that this influence is accurately captured by the simplified model through the application of Eq.(23). The strength reduction factors for the structural steel k_{yT} and reinforcing bars k_{bT} can be determined according to the temperatures of these respective parts. The reduction factors for structural steel at

530 elevated temperatures provided in EC3 (2005) are in fact the same as those specified for reinforcing
 531 bar in EC2 (2004). Similar values are given in CECS200 (2006), and presented below:

$$532 \quad k_{bT} = k_{yT} = \begin{cases} 1.0 & 20^\circ\text{C} \leq T \leq 300^\circ\text{C} \\ 1.24 \times 10^{-8} T^3 - 2.096 \times 10^{-5} T^2 + 9.228 \times 10^{-3} T - 0.2168 & 300^\circ\text{C} < T < 800^\circ\text{C} \\ 0.5 - T / 2000 & 800^\circ\text{C} \leq T \leq 1000^\circ\text{C} \end{cases} \quad (23)$$

533 For axially loaded STCRC slender columns, the load-bearing capacity can be obtained as follows:

$$534 \quad N_u(T) = \chi N_0(T) \quad (24)$$

535 where χ is the buckling reduction factor. Parametric studies were conducted to identify the influences
 536 of slenderness ratio, heating time and cross-sectional dimensions on the buckling capacity of STCRC
 537 columns; a total of 294 numerical parametric results were generated, based upon which the following
 538 equation is proposed for the prediction of the buckling reduction factor χ :

$$539 \quad \chi = \begin{cases} 1 & \lambda \leq 12 \\ 1 - 0.006(\lambda - 12) \left(1.0 + \frac{0.16t}{D} \right) & \lambda > 12 \end{cases} \quad (25)$$

540 where t is the heating time in hours, D is the cross-sectional diameter in meters, $\lambda=4L/D$ is the
 541 slenderness ratio and L is the effective column length.

542 The load-bearing capacity of STCRC columns subjected to the ISO 834 standard fire can hence be
 543 predicted using Eqs (24) and (28). The fire resistance and failure temperature of the STCRC columns
 544 were predicted using the proposed method; the test and predicted values are compared in Table1,
 545 confirming the accuracy of the proposals. Comparisons of the predicted load-bearing capacities
 546 between the simplified method and the FE parametric results are shown in Fig.26. The mean and
 547 standard deviation of the ratio of the simplified design method-to-FE model predicted cross-section
 548 capacity are 0.997 and 0.136, respectively, whereas the mean and standard deviation of the ratio of
 549 the simplified design method-to-FE model predicted buckling capacity are 0.994 and 0.106,
 550 respectively.

551 The limits of the validity of the above design method are as follows: $D=0.2 - 1.5$ m, $\lambda \leq 60$, $f'_c=24$
 552 N/mm² - 50 N/mm², $f_y=235$ N/mm² - 420 N/mm², $f_b=335$ N/mm² - 500 N/mm², $\alpha_s=2\% - 4\%$, $\alpha_b=2\%$
 553 - 5%.

Conclusions

An experimental study was first conducted to investigate the fire performance of circular STCRC columns subjected to the ISO 834 standard fire. Full-scale fire tests were carried out on four STCRC columns and one CFST column to determine the fundamental behavior of these columns. The furnace and specimen temperatures, displacement versus time curves and fire resistances were presented and discussed. It was found that the temperatures across the cross-sections of the STCRC and CFST columns decrease from the outer steel tube to the center of the concrete, and that the temperature of the concrete remains at around 100 °C for some time, due to the energy absorption associated with the moisture evaporation and migration. The development of the deformation of the STCRC columns resembled that of the CFST column. The STCRC columns under combined thermal and structural loading failed by global buckling, accompanied by local buckling. The inner concrete remained largely intact due to the presence of the outer steel tube, despite some crushing of the concrete. The load ratio had a significant influence on the fire resistance of the STCRC columns, but compared to the CFST column, the STCRC columns provided much higher fire resistance, owing to the contribution of the embedded reinforcing bars.

Complementary to the experimental study, a nonlinear finite element (FE) study was conducted to investigate the behavior of reinforced concrete columns confined by circular steel tubes subjected to the ISO 834 standard fire. The main input parameters for the FE model were discussed, carefully selected and shown to result in good predictions of observed physical behavior; these chosen parameters will serve as useful reference values for others in future work. The fire behavior of a series of tested STCRC columns and CFST columns exposed to fire were predicted by the means of the developed FE model. Good agreement was achieved between the FE model and the test results in terms of the specimen temperatures, deformation-time responses and fire resistance. Parametric studies were then performed using the validated FE model to generate data over a wide range of STCRC column cross-section geometries, heating times and slenderness ratios. A simplified design method was proposed for predicting the equivalent temperature, load-bearing capacity and fire resistance of STCRC columns. The accuracy of the proposals was verified through comparisons with the test and numerical results.

582 The limits of this study are as follows: $D=0.2 - 1.5$ m, $\lambda \leq 60$, $f'_c=24$ N/mm² - 50 N/mm², $f_y=235$
583 N/mm² - 420 N/mm², $f_b=335$ N/mm² - 500 N/mm², $\alpha_s=2\%$ - 4%, $\alpha_b=2\%$ - 5%.

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588 **Notation**

589 The following symbols are used in this paper:

a =thickness of fire protection

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_b =diameter of reinforcing bars

D =outer diameter of cross-section

D_c =outer diameter of concrete core

e =load eccentricity

E_b =Young's modulus of reinforcing bars

E_c =Young's modulus of concrete

E_s =Young's modulus of structural steel

$f_{bi,CT}$ =bi-axial compressive strength of concrete at elevated temperature

f_{by} =yield strength of reinforcing bars

f_{bu} =ultimate tensile strength of reinforcing bars

f_c =concrete compressive strength

f'_c =concrete cylinder strength

f_{cu} =concrete cube strength

f_{sy} =yield strength of structural steel

f_{su} =ultimate tensile strength of structural steel
 L =length of column
 n =load ratio
 N_0 =cross-section capacity of composite column at ambient temperature
 $N_0(T)$ =cross-section capacity of composite column at elevated temperature
 N_f =applied axial load
 N_u =ultimate capacity of composite column at ambient temperature
 $N_u(T)$ =ultimate capacity of composite column at elevated temperature
 t_s =wall thickness of steel tube
 T =temperature
 $T_{c,eq}$ =equivalent temperature of concrete
 T_b =temperature of reinforcing bar
 T_s =temperature of steel tube
 α =thermal expansion coefficient
 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_{cth} =thermal strain of concrete
 ε_{ctr} =transient strain of concrete
 ν_c =Poisson's ratio of concrete at ambient temperature
 ν_{ct} =Poisson's ratio of concrete at elevated temperature
 ν_s =Poisson's ratio
 χ =buckling reduction factor

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772

Fig.1 Details of the test specimens: (a) STCRC-1 to STCRC-3 (case a); (b) STCRC-4 (case b); (c) CFST; (d) cross-section of the STCRC columns; and (e) cross-section of the CFST column (unit: mm).

Fig.2 Layout of thermocouples in the test specimens: STCRC1-STCRC4: (a) at hole no. 1; (b) at hole no. 2; (c) at hole no. 3; and CFST: (d) at hole no. 1; (e) at hole no. 2; (f) at hole no. 3 (unit: mm).

Fig.3 General view of test rig and specimen: (a) overview of the test rig with dimensions in mm; (b) specimen prior to testing; (c) schematic view of top boundary condition; and (d) real top boundary condition.

Fig.4 Failure modes of test specimens: general view: (a) STCRC-1; (b) STCRC-2; (c) STCRC-3; (d) STCRC-4; and (e) CFST; and typical local buckling: (f) STCRC and (g) CFST.

Fig.5 Failed specimens upon removal of steel tube.

Fig.6 Furnace temperature versus time curves at different locations.

(a)

(b)

Fig.7 Average furnace temperature versus time curves: (a) comparison to standard fire curve; and (b) deviation from standard fire curve.

(a)

(b)

(c)

Fig.8 Temperatures of specimen at different heights (STCRC-4): (a) steel tube; and (b) - (e) concrete.

(a)

(b)

(c)

(d)

(e)

Fig.9 Measured temperatures of specimens through cross-sections: (a) STCRC-1; (b) STCRC-2; (c) STCRC-3; (d) STCRC-4 and (e) CFST.

(a)

(b)

(c)

(d)

(e)

Fig.10 Displacement versus time curves of specimens during fire: (a) STCRC-1; (b) STCRC-2; (c) STCRC-3; (d) STCRC-4; and (e) CFST.

(a)

(b)

Fig.11 Influences of parameters on displacement-time response: (a) load ratio; (b) frequency of breaks to the longitudinal continuity of the steel tube; and (c) member type.

(a)

(b)

Fig.12 Finite element model of STCRC columns: (a) schematic view and (b) cross-section.

(a)

(b)

Fig.13 Comparisons between the EC3 model and Lie model: (a) thermal expansion coefficient; and (b) relative thermal elongation.

(a)

(b)

(c)

(d)

(e)

Fig.14 Comparisons between the FE predicted and test temperatures of STCRC columns: (a) STCRC-1; (b) STCRC-2; (c) STCRC-3; (d) STCRC-4; and (e) CFST.

(a)

(b)

(c)

(d)

Fig.15 Comparisons between the FE predicted and test temperatures of plain concrete-filled steel tubular columns (Lie and Chabot 1992): (a) C11; (b) C21; (c) C29; and (d) C60.

(a)

(b)

Fig.16 Comparisons between the FE predicted and test temperatures of reinforced concrete-filled steel tubular columns (Chabot and Lie 1992): C49: (a) steel tube and concrete; and (b) reinforcing bar.

(a)

(b)

(c)

(d)

Fig.17 Comparisons between the FE predicted and test deformation-time curves of STCRC columns: (a) STCRC-1; (b) STCRC-2; (c) STCRC-3; (d) STCRC-4; and (e) CFST.

(a)

(b)

(c)

(d)

(e)

(f)

Fig.18 Comparisons between the FE predicted and test deformation-time curves of plain concrete-filled steel tubular columns (Lie and Chabot 1992; Han et al. 2013): (a) C-02 and C-17; (b) C-05 and C-22; (c) C-31 and C-34; (d) C-32 and C-35; (e) C1-1 and C2-1; and (f) C1-2 and C2-2.

(a)

(b)

(c)

(d)

Fig.19 Comparisons between the FE predicted and test deformation-time curves of reinforced concrete-filled steel tubular columns (Chabot and Lie 1992; Espinos et al. 2015): (a) C48; (b) C49; (c) C1; and (d) C3.

Fig.20 Comparison between FE predicted and test fire resistances.

(a)

(b)

(c)

(d)

(e)

(f)

(g)

(h)

Fig.21 Influences of parameters on the fire resistance: (a) load ratio; (b) cross-section dimension; (c) slenderness ratio; (d) concrete strength; (e) yield strength of steel; (f) yield strength of reinforcement; (g) steel tube to concrete area ratio; and (h) reinforcement ratio.

Fig.22 Comparison of predicted temperatures of steel tube between simplified methods and FE model ($D=600$ mm, $\alpha_s=3.0\%$).

(a)

(b)

Fig.23 Influence of thermal resistance (TR) on the concrete temperature ($\alpha_s=3\%$): (a) $D=600$ mm; and (b) $D=1200$ mm.

Fig.24 Comparison of predicted reduction factor for the concrete strength between simplified method and FE model.

(a)

(b)

Fig.25 Influences of thermal gradient on the load-bearing capacities of circular STCRC columns under: (a) uniform temperature and (b) ISO 834 standard fire.

(a)

(b)

Fig.26 Comparison between predicted load-bearing capacities using the simplified design method and the FE model: (a) cross-section capacity; and (b) buckling capacity.

Table 1 Details of test specimens

Column no.	D (mm)		t_s (mm)		L (mm)	α_s (%)	Reinforcing bars	α_b (%)	N_f (kN)	n	Case	Fire Resistance (min)		
	Nominal	Measured	Nominal	Measured								Test	FE	Eq.
STCRC-1	300	299.2	2.75	2.53	3810	3.38	8ø20	3.68	1340	0.25	a	117	98	104
STCRC-2	300	299.5	2.75	2.53	3810	3.38	8ø20	3.68	1800	0.33	a	83	69	79
STCRC-3	300	298.9	2.75	2.53	3810	3.39	8ø20	3.68	2240	0.42	a	50	41	60
STCRC-4	300	299.1	2.75	2.53	3810	3.38	8ø20	3.68	2240	0.42	b	54	41	60
CFST	300	299.3	6.00	5.67	3810	7.58	-	-	2168	0.42	-	23	26	-

Table 2 Measured properties of the structural steel

Column no.	t_s (mm)	f_{sy} (N/mm ²)	f_{su} (N/mm ²)	E_s ($\times 10^5$ N/mm ²)	ν_s
STCRC1-STCRC4	2.53	291.3	409.5	1.955	0.24
CFST	5.67	264.6	435.3	1.795	0.26

Table 3 Measured properties of the reinforcing bars

Steel type	d_b (mm)	f_{by} (N/mm ²)	f_{bu} (N/mm ²)	E_b ($\times 10^5$ N/mm ²)
Hot-rolled plain	6.8	435.4	668.8	1.930
Hot-rolled ribbed	20.0	357.4	570.5	1.838

Table 4 Measured properties of the concrete

f_{cu} (N/mm ²)	E_c ($\times 10^4$ N/mm ²)	Ages of concrete (days)
33.7	2.518	28
73.2	3.850	518

Table 5 Details of CFST columns

Column no.	Sources	D (mm)	t_s (mm)	Reinforcing bars	Concrete cover (mm)
C02	Lie and Chabot (1992)	141.3	6.55	-	-
C11		219.1	4.78	-	-
C21		273.1	5.56	-	-
C25		323.9	6.35	-	-
C29		355.6	12.7	-	-
C60		406.4	12.7	-	-
C48	Chabot and	273.1	6.35	4 ϕ 19.5 mm	23
C49	Lie (1992)	273.1	6.35	4 ϕ 19.5 mm	23

Table 6 Details of plain concrete-filled steel tubular columns

Column no.	Sources	D (mm)	t_s (mm)	L (mm)	e (mm)	Boundary conditions	f'_c (MPa)	f_y (MPa)	N_f (kN)	a (mm)	Fire resistance (min)	
											Test	FE
C-02	Lie and Chabot (1992)	141.3	6.55	3810	0	Fixed -fixed	33.1	350	110	0	55	65
C-04		141.3	6.55	3810	0		31.0	350	131	0	57	53
C-05		168.3	4.78	3810	0		32.7	350	150	0	76	86
C-08		168.3	4.78	3810	0		35.5	350	218	0	56	71
C-09		168.3	6.35	3810	0		35.4	350	150	0	81	90
C-13		219.1	4.78	3810	0		32.3	350	384	0	102	111
C-17		219.1	8.18	3810	0		31.7	350	525	0	82	85
C-20		273.1	5.56	3810	0		28.6	350	574	0	112	169
C-21		273.1	5.56	3810	0		29.0	350	525	0	130	183
C-22		273.1	5.56	3810	0		27.2	350	1000	0	70	85
C-23		273.1	12.7	3810	0		27.4	350	525	0	143	170
C-25		323.9	6.35	3810	0		27.6	350	699	0	145	160
C-26		323.9	6.35	3810	0		24.3	350	1050	0	93	93
C-29		355.6	12.7	3810	0		25.4	350	1050	0	170	236
C-31		141.3	6.55	3810	0		30.2	300	80	0	82	75
C-32		141.3	6.55	3810	0		34.8	300	143	0	64	51
C-34		219.1	4.78	3810	0		35.4	300	500	0	111	95
C-35		219.1	4.78	3810	0		42.7	300	560	0	108	100
C-37		219.1	8.18	3810	0		28.7	350	560	0	102	72
C-40		273.1	6.35	3810	0		46.5	350	1050	0	106	145
C-42		273.1	6.35	3810	0		55.4	350	1050	0	90	111
C-44		273.1	6.35	3810	0		38.7	350	715	0	178	175
C-45		273.1	6.35	3810	0		38.2	350	712	0	144	173
C-50		323.9	6.35	3810	0		42.4	300	820	0	234	317
C-55		355.6	12.7	3810	0		40.7	300	965	0	274	361
C1-1	Han et al. (2013)	478	8	3810	0	Pinned -pinned	39.6	293	4700	0	29	32
C1-2		478	8	3810	71.7		39.6	293	2200	0	32	30
C2-1		219	5	3810	32.9		39.6	293	450	0	17	13
C2-2		219	5	3810	65.7		39.6	293	300	0	18	17
C2-3		219	5	3810	0		39.6	293	960	15	132	94
C2-4		219	5	3810	0		39.6	293	960	25	175	156

Table 7 Details of reinforced concrete-filled steel tubular columns

Column no.	Sources	D (mm)	t_s (mm)	Reinf.	e/D	L (mm)	Boundary conditions	f'_c (MPa)	f_y (MPa)	f_b (MPa)	N_f (kN)	Fire resistance (min)	
												Test	FE
C48	Chabot and Lie (1992)	273.1	6.35	4ø19.5	0	3810	Fixed	46.7	350.0	400.0	1050.0	188	154
C49		273.1	6.35	4ø19.5	0	3810	-fixed	47.0	350.0	400.0	1900.0	96	88
C1	Espinosa et al. (2015)	193.7	8	6ø12	0.5	3210	Pinned -pinned	36.4	359.1	512.4	186.7	26	32
C2		273.0	10	6ø16	0.5	3210		37.6	369.7	553.5	387.5	30	51
C3		193.7	8	6ø12	0	3210		43.2	359.1	512.4	535.6	29	27
C5		193.7	8	6ø16	0.75	3210		35.8	359.1	553.5	152.4	29	41