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Fire performance of concrete-filled steel tubular columns strengthened by CFRP

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Abstract. With the increasing use of concrete-filled steel tubes (CFST) as structural members, there is a growing need to provide suitable measures for possible strengthening or repair of these kinds of structural elements. Fibre reinforced polymer (FRP) jacketing is a recent method and is particularly attractive in which it does not significantly increase the section size, and is relatively easy to install. Thus, it can be used to enhance strength and/or ductility of CFST members. Very little information is available on the performance of FRP-strengthened CFST members under fire conditions. This paper is an attempt to study the fire performance of CFST columns strengthened by FRP. The results of fire endurance tests on FRP-strengthened circular CFST columns are presented. Failure modes of the specimens after exposure to fire, temperatures in the cross section, axial deformation and fire resistance of the composite columns are analysed. It is demonstrated that the required fire endurance can be achieved if the strengthened composite columns are appropriately designed.

Keywords: concrete-filled steel tubes (CFST); fibre reinforced polymer (FRP); strengthening; columns; insulation; confinement; fire endurance

1. Introduction

Concrete-filled steel tubular (CFST) columns have been widely used in civil engineering in Australia, China, Japan, USA, as well as in many other countries. This is largely due to the structural and economical benefits resulting from the ideal combination of the advantages of both steel and concrete (Uy and Patil 2006, Zhao and Han 2006, Han 2007, Lam and Gardner 2008).

With the increasing use of concrete-filled steel tubes as structural members, there is a growing need to provide suitable measures for the possible strengthening or repair of these kinds of elements. Damage resulting from fire or elevated temperatures is one of the growing risks for structural elements, which has been highlighted by several recent disasters in China. The first one involved two CFST columns which were damaged by high-temperature molten glass leaking from a large glass-melting furnace in the 1990s in China (Lin *et al*. 1997). Both the columns were finally repaired by a section enlargement approach. Another recent case occurred in a textile mill where CFST columns were used in a workshop building of the mill. This workshop building was almost destroyed by a fire in early 2007. A general internal view of the building after the disastrous fire and one of the fire-damaged CFST columns is

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(a) A general internal view

(b) Damaged CFST column

Fig. 1 A workshop building damaged in fire (2007, China)

shown in Fig. 1(a) and 1(b), respectively. This building has been repaired recently. Although no effort was made to enhance the load-carrying capacity of the fire-damaged CFST columns, lighter roof and ceiling has been adopted in the repair scheme with an aim to reduce the column loads. Otherwise, fibre reinforced polymer (FRP) composites may be considered as a very suitable repair approach to enhance the load-carrying capacity of the columns.

In recent years, there has been an increasing interest in the use of FRP materials in retrofitting/repairing existing concrete or steel structures (Teng *et al*. 2002, Tao and Yu 2006, Zhao and Zhang 2007). This is attributed to a host of advantages of FRPs, including high strength-to-weight ratio, high corrosion resistance and ease of installation.

More recently, some researchers have investigated the behaviour of strengthened CFST stub columns, where FRP materials were used for confinement purposes (Wang *et al*. 2004, Xiao *et al*. 2005, Teng 2006, Tao *et al*. 2007b, Wang *et al*. 2009). These studies highlighted the effectiveness of the FRP wraps in strengthening composite columns. The authors have been engaged in studies of the behaviour of fire-damaged CFST columns repaired with FRP wraps, where carbon fibre reinforced polymer (CFRP) composites were used to fabricate the confining jackets. Tao and Han (2007) and Tao *et al*. (2007a) conducted monotonic tests on fire-damaged CFST columns, beams and beam-columns repaired with CFRP wraps, respectively. It demonstrated that the CFRP composites were effective in enhancing the load-carrying capacity of the columns and the beam-columns, whilst the strengthening effect for beams was not obvious if only unidirectional FRP sheets were used to confine the beams. Tao *et al*. (2008) conducted twenty tests to evaluate the cyclic performance of the fire-damaged CFST beam-columns repaired with CFRP wraps. It was found that the load-carrying capacity and the stiffness of the repaired specimens increased with the increasing number of CFRP layers, whilst the ductility increased only slightly. More recently, Wang *et al*. (2009) performed a nonlinear analysis of FRP-strengthened CFST columns under axial compression using ABAQUS software (ABAQUS 2007), and the confinement mechanisms of FRP and steel tube on the column behaviour were investigated. It was found that the confinement from the steel tube is minor for a FRP-strengthened CFST column prior to the occurrence of the FRP rupture, and after that, the confinement is provided by the steel tube only and a significant increase of the confinement from the steel tube occurs.

From the above studies, it is obvious that the FRP materials have the potential to be used in strengthening/repairing CFST columns. It is well-known that FRP materials, however, are susceptible to deterioration of mechanical properties and combustion when exposed to fire or elevated temperatures (Han *et al*. 2006, Chowdhury *et al*. 2008, Ji *et al*. 2008, Williams *et al*. 2008). The polymer resin will become rubbery and viscous if the temperature ranges from 65 to 150°C, and will be susceptible to combustion at temperatures above 400°C (Ji *et al.* 2008). As a result, there is a need to understand the performance of FRP-strengthened CFST columns in fire, which is critical to ensure an efficient and safe use of FRP composites in strengthening/repairing CFST columns.

Although a number of studies have been conducted on the fire behaviour of FRP-strengthened reinforced concrete columns recently (Bisby *et al.* 2005a, 2005b, Han *et al.* 2006, Chowdhury *et al.* 2007, Kodur *et al.* 2006, 2007, Liu *et al.* 2009, Hawileh *et al.* 2009), there is currently a lack of understanding of the performance of FRP-strengthened CFST columns under fire. Two fire endurance test results on FRP-strengthened circular CFST columns are thus presented in this paper, with an aim to understand the heat transfer and fire endurance of the strengthened composite columns.

2. Experimental investigation

2.1. General

Two strengthened CFST columns shown in Table 1 were prepared and tested. One of the columns was loaded axially and designated as CCFT-1, and another column was designed to be subjected to eccentric loading and designated as CCFT-2. Figs. 2 and 3 show the cross-section and elevation details of the columns, respectively. The steel tubes were chosen to have an outer diameter (D) of 325 mm and a thickness (t_s) of 5 mm, where these sizes were determined based on the loading capacity of the hydraulic jack. The total length of the specimens was 3810 mm. As they were slender columns, strengthening effects could be achieved more effectively by using bidirectional CFRP. Thus, the CFRP strengthening in this test program was designed by firstly bonding two layers of unidirectional carbon fibre sheets with the fibres oriented in the longitudinal direction of the columns, and then by wrapping two layers of the same carbon fibre sheets with the fibres oriented in the hoop direction. Fire protection was applied to the exterior of the FRP with an insulation thickness of 5 mm.

2.2. Material properties

Cold-formed steel tubes were used in the fabrication of the specimens. Tensile tests on steel coupons cut from the original steel tubes were conducted. The measured properties of the steel tubes were: elastic modulus E_s = 209 kN/mm^2, Poisson's ratio ν_s = 0.298; yield strength f_y = 383.2 N/mm^2, yield strain ε_y = 1988 με, ultimate strength f_u = 434 N/mm^2, and ultimate strain ε_u = 190000 με.

Both tubes were filled with one batch of self-consolidating concrete. The maximum size of coarse aggregate was 15 mm. In order to increase the slump and make the concrete more workable, fly ash and water reducer were added in the concrete mix. The mix proportions were as follows: cement: 361 kg/m^3; fly ash: 168 kg/m^3; water: 176 kg/m^3; sand: 795 kg/m^3; coarse aggregate: 896 kg/m^3; additional high-range

Table 1 Specimen details

Specimen label	Tube diameter D (mm)	Tube thickness t_s (mm)	Load eccentricity e (mm)	Insulation thickness a (mm)	Axial load N_o (kN)	Measured fire endurance (min)	Calculated fire endurance (min)
CCFT-1	325	5	0	5.6	2200	>162	163.6
CCFT-2	325	5	81	5	1710	77	80.1

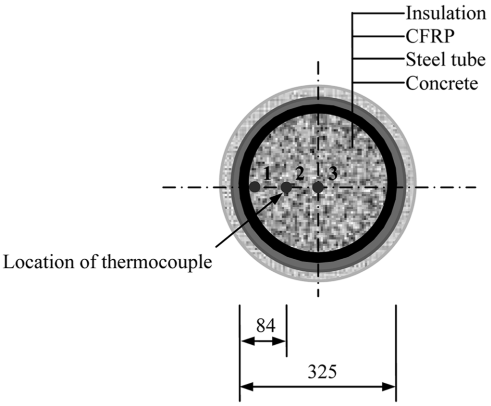


Fig. 2 Cross-section of strengthened, insulated CFST column (Unit: mm)

water reducer: 5 kg/m^3. An L-box was used to measure rheological parameters of fresh concrete (Han and Yao 2004). The fresh properties of the SCC mixture were as follows: slump flow: 225 mm; flow speed: 21 mm/s; and flow distance: 610 mm. To determine the compressive strength of concrete, six 150 mm cubes were cast and cured in conditions similar to the related specimens. The measured average cube strength (f_{cu}) at 28 days and at the time of the fire tests was found to be 26.0 N/mm^2 and 37.6 N/mm^2, respectively.

Unidirectional carbon fibre sheets, which were commercially available, were used to strengthen the composite columns. The mechanical properties of the cured CFRP determined from tensile testing of flat coupons according to ASTM D3039 (2006), were calculated on the basis of the nominal thickness of 0.167 mm for the fibre sheets. The measured average elastic modulus E_f was 240 kN/mm^2 with an ultimate tensile strength of 4480 N/mm^2.

A commercially available spray material was used to fabricate the fire protection coating. This type of material has been widely used in protecting steel members in China. The material tests according to CECS24:90 (1990) show that it had a thermal conductivity (λ_p) of 0.0777 W/(m•K), a specific heat (c) of 1.01 × 10^3 J/(kg•K), and a density (ρ) of 305 kg/m^3.

2.3. Specimen preparations

In fabricating a specimen, the steel tube was cut and machined to the required length first (Fig. 3), and then tack welded with two steel end plates of 40 mm thickness before filling the tube with concrete. Twelve bolt holes were drilled at each end plate, which were used to fix the specimen to the loading system of the furnace. In order to enhance the stiffness of the end plates, four stiffeners were welded to each end of the specimen as shown in Fig. 3. A circular hole of 250 mm in diameter was drilled on the top end plate and used to pour the concrete. Two semi-circular holes with 20 mm diameter, located at the junctions between the tube and the top plate as well as the bottom plate, were drilled in the section wall. They were provided as vent holes for the water vapour pressure produced during the fire testing.

Self-consolidating concrete was used, and filled in layers without any vibration shown in Fig. 4(a). After the concrete was cured for three days, the top loose layer of mortar for each specimen was moved. The two specimens were placed upright to air-dry for about one week. Then a layer of high-strength epoxy mortar was applied on the top of each specimen. and the top hole was repaired by welding a steel plug (cut from the top end plate earlier) to ensure the flushness of the top surface. The same epoxy resin

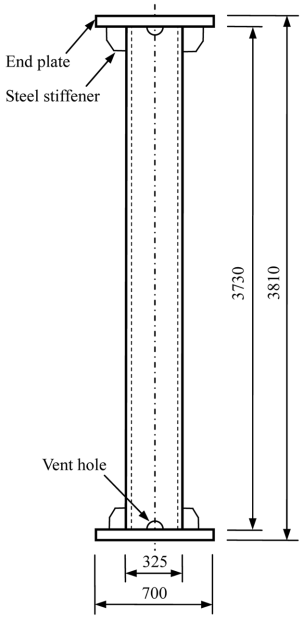


Fig. 3 Elevation details of CFST columns prior to strengthening (Unit: mm)

used to install the CFRP system was used to make the epoxy mortar. Based on the test results conducted by Wang and Xu (1997), the adopted mix proportion by weight was: epoxy:cement:sand:xylene:dibutyl ester:ethylenediamine = 10 : 10 : 15 : 2 : 1.5 : 1. The target compressive strength for this mortar was 50 N/mm^2 at 28 days of curing which was much higher than that of the concrete core. It was expected that this end treatment had no significant influence on the column's performance since the depth of the mortar was only about 5 mm and the ends of the column were not exposed to fire. This was confirmed by the fact that no local failure occurred at either end of a specimen.

Before wrapping of the carbon fibre sheets, the concrete infill was cured for 28 days. Prior to the wrapping operation, the specimen surface was ground, cleaned with water and left to dry. Hand-mixed epoxy adhesive was first applied evenly on the steel surface. The sheets were presaturated with resin and then applied to the resin-wetted steel surface. A roller was used to squeeze out extra epoxy and any entrapped air. To ensure adequate bond and continuity of the jacket, a 150 mm overlap was used for the fibre sheets.

Prior to the fire testing, a supplemental fire insulation system was applied to the exterior of the FRP, where a coating material was spray-applied in a thin layer along the entire length of each column [Fig. 4(c)]. Since the bond between the coating material and the FRP jacket is critical to ensure the effectiveness of the fire insulation, a patented two-component insulation system (VG insulation and EI-R coating) and a unique spray-applied cementitious mortar-based fire protection system have been used by Williams *et al.* (2008) and Chowdhury *et al.* (2007) to protect FRP-strengthened reinforced concrete T-beams and columns, respectively. For the spay material used in this test program, it has been proved that it can adhere well to steel, concrete and masonry. Its bond behaviour with FRP under fire conditions, however, remains unknown. In order to ensure a strong bond of the insulation coating with the FRP material, steel wire mesh was cut and shaped to closely fit the contour of the section before the insulation material was sprayed, as shown in Fig. 4(b). The woven wire mesh had a 0.66 mm diameter wire strands with mesh openings of 12 × 12 mm. A similar approach has been adopted by Hu *et al.* (2007).

2.4. Instrumentation and test setup

To monitor the heat transfer in the specimens during the fire testing, three thermocouples were embedded in each specimen at the mid-height, where the thermocouple locations are schematically shown in Fig. 2. Since only a thin layer of fire insulation coating was applied on the FRP jacket, no attempt was made to measure the temperatures at the epoxy interface during the fire tests. However, the temperatures at this location can be evaluated from those recorded at the inner surface of the steel tube (Point 1 shown in Fig. 2), since the temperatures at the inner and outer surfaces of the steel tube were expected to be virtually the same.

The fire tests were conducted in a furnace specially built for testing loaded columns in Tianjin, China (Han *et al.* 2003). The furnace chamber has a floor area of 2600 × 2600 mm with a height of 4,000 mm. The length that was exposed to fire for each column was approximately 3,000 mm in the testing. The axial deformation of each specimen was determined by measuring the displacement of the loading jack during the fire testing.

2.5. Test procedure

The specimens were installed in the furnace by bolting the end plates to the test frame loading head at the top and hydraulic jack at the bottom. The end conditions of the columns were fixed-pinned for all tests. The columns were subjected to a sustained axial compression load N_o, which represented about 50% of the calculated ultimate capacity N_u. The applied axial load ratio (N_o/N_u) was 0.485 for CCFT-1 and 0.514 for CCFT-2. The values of N_u were determined by using the mechanics model presented by Zhuang (2006), where measured material properties were used in the calculation.

During the fire tests, the furnace heating was controlled as closely as possible to the ISO- 834 standard fire curve (ISO- 834 1999). According to this Standard, a column is considered to have failed if the

(a) Filling concrete (b) Wrapping CFRP and steel wire mesh (c) Installed in the furnace with insulation coating

Fig. 4 Preparation of test specimens

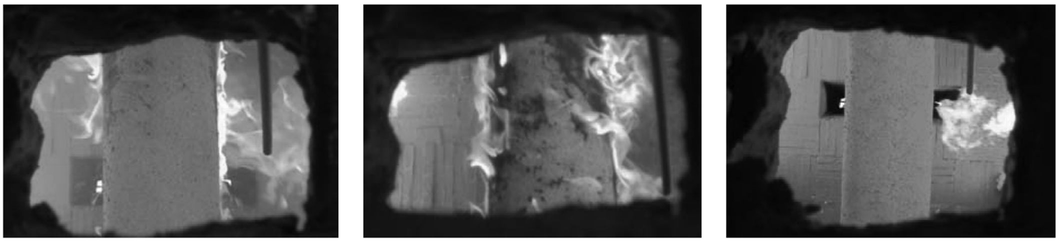
column has contracted axially by 0.01 L mm and the rate of contraction has reached 0.003 L mm/min, where L is the length of the column in mm.

3. Experimental results and discussion

3.1. Test observations

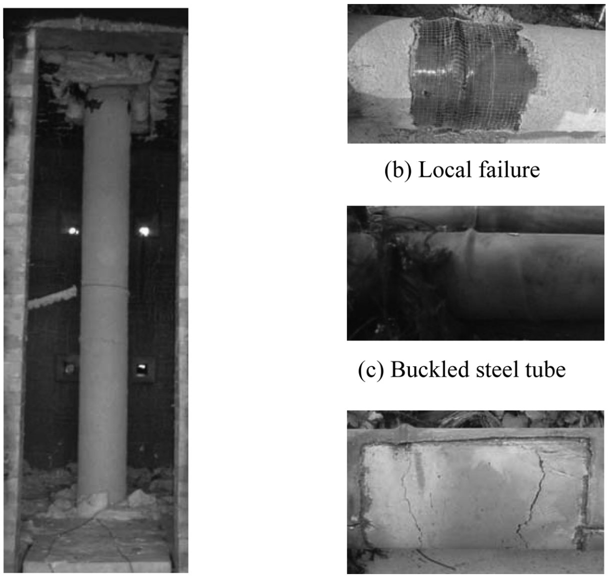
Several small view ports were located around the furnace walls which allowed observation during the fire testing. With a thin insulation coating, some minor cracking appeared on the coating at about 6 min fire exposure for the axially loaded specimen CCFT-1. The composite material then began to burn at the locations of cracking as shown in Fig. 5(a). The flame surrounded the entire surface of the column at about 16 min [Fig. 5(b)]. The minor cracking was mainly resulted from the shrinkage of the insulation coating at elevated temperatures. In general, the thinner the coating, the earlier the cracking will occur (Hu *et al*. 2007). After 28 min of fire exposure, several bulges with a maximum height of 15 mm along the column could be observed. This was attributed to the pressure resulting from water vapour from the core concrete. Since the vapour was entrapped by the FRP jacket, no evaporation of water, however, could be observed directly from the view ports. At 46 min fire exposure, it was found that the FRP almost stopped burning [Fig. 5(c)], suggesting the fully consumption of the combustible resin. Ji *et al*. (2008) conducted an experimental study of the FRP tube encased concrete cylinders exposed to fire. The elapsed time of resin burning for the specimens without fireproof coating presented by Ji *et al*. (2008) was only 8 min. Compared with the current test results, it seems that even a thin insulation coating can effectively postpone the burning of the resin, and thus possibly delay the failure of the FRP jackets. The test on the specimen CCFT-1 ended at 162 min fire exposure owing to the overheating of the hydraulic loading system. Thus, the fire resistance was not attained. During the fire exposure, no apparent deflection was observed, and no insulation material delaminated from the test specimen except at the location of local failure [Fig. 6(a)], which demonstrated the good bond between the coating and the specimen.

As far as the eccentrically loaded specimen CCFT-2 is concerned, the observed phenomena were generally the same as those described above for the specimen CCFT-1. The cracking of the insulation coating and the burning of the resin, however, firstly occurred at the tension side of the specimen. Another notable difference was that apparent deflection could be observed at about 30 min fire exposure due to the bending influence. Failure of this column occurred at the quarter point near the top end (shown in Fig. 7(a)) after 77 min of exposure to fire.



(a) t=6 min (b) t=16 min (c) t=46 min

Fig. 5 Specimen CCFT-1 at different times of fire exposure

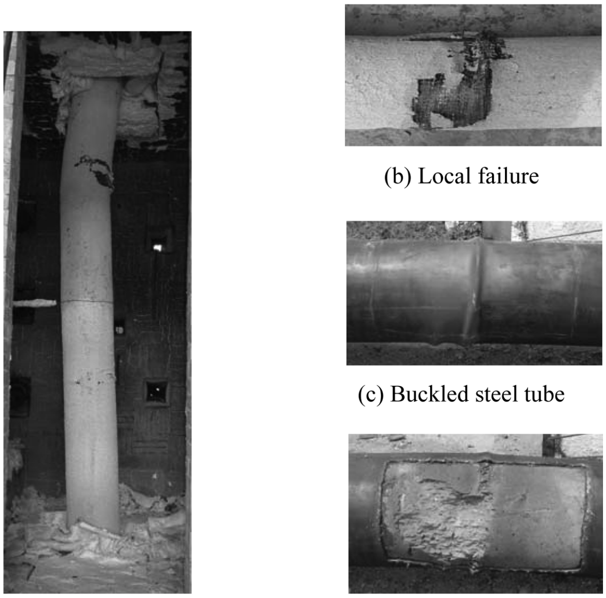


(b) Local failure

(c) Buckled steel tube

(a) General view (d) Cracked concrete

Fig. 6 Specimen CCFT-1 after fire test



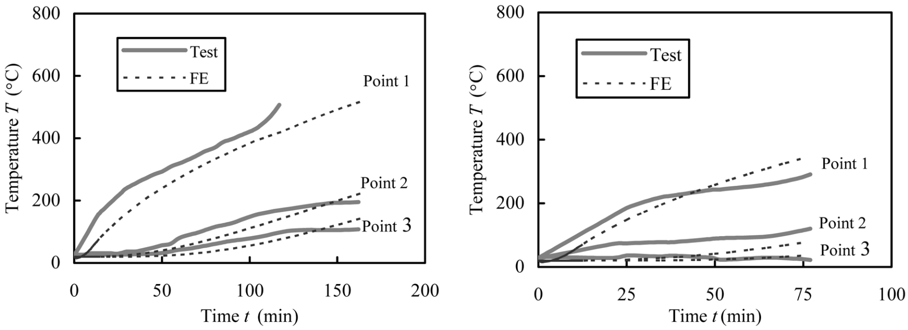
(b) Local failure

(c) Buckled steel tube

(a) General view (d) Crushed concrete

Fig. 7 Specimen CCFT-2 after fire test

Although no signs of impending failure were observed for the specimen CCFT-1 when the test ended, local failure can be found near its top end after the fire test, as shown in Fig. 6(a). For both the test specimens, it was found that the steel tubes buckled as shown in Figs. 6 and 7, and the concrete



(a) Specimen CCFT-1 (b) Specimen CCFT-2

Fig. 8 Temperatures recorded at different locations as a function of time

cracking or crushing was observed at the locations where tube buckling had occurred. Owing to the protection of the insulation coating, the carbon fibres remained intact after removing the coating material. The resin, however, was burnt with some carbonized residues left on the fibre surface.

3.2. Temperatures

Temperatures recorded at the three measurement locations during the fire endurance testing are shown in Fig. 8. Since concrete has a relatively low thermal conductivity, the temperatures of the measurement points 2 and 3 within the concrete were much lower than the temperature recorded at the inner surface of the steel tube (Point 1).

As can be seen from Fig. 8, the temperature at the measurement point 1 increased rapidly to almost 200°C within the first 25 min of fire exposure. Although no thermocouples were installed to measure the temperature at the FRP surface, it can be presumed that the temperature there was a little higher than that measured at point 1 owing to the early ignition of the FRP and its proximity to the insulation coating. It is clear that the effectiveness of the FRP jackets had been influenced early during fire exposure.

Heat transfer analysis was performed using the general-purpose finite element analysis software ABAQUS (ABAQUS 2007). A three-dimensional finite element (FE) model was developed, where eight-node continuum (DC3D8) heat transfer elements were used to model the concrete core, and four-node shell (DS4) heat transfer elements were used to model other components, including the steel tube, the FRP jacket and the insulation coating. The FE model of a typical strengthened CFST column is shown in Fig. 9, where the longitudinal CFRP and transverse CFRP were modelled separately. The "TIE" constraint available in ABAQUS was used to define the interactions across: 1) insulation coating and transverse CFRP; 2) transverse CFRP and longitudinal CFRP; 3) longitudinal CFRP and steel tube. Without considering the thermal resistance between the contact surfaces, the "TIE" constraints enforced equal temperatures for the surfaces in thermal contact. To simulate the interaction between the steel tube and concrete, hard contact in the normal direction and Coulomb friction model in the tangential direction were adopted. According to Lu *et al.* (2009), a heat conductance of 100 W/(m^{2}K) was used to represent the heat resistance in this interface. Although the introduction of a friction coefficient and bond strength between the steel tube and concrete has no influence on the heat transfer analysis, they may affect the structural behaviour of the composite column in the following stress analysis. Based on a sensitivity analysis, Lu *et al.* (2009) concluded that the bond strength could be neglected and a friction coefficient of 0.18 could be used. Therefore, this suggestion was followed in this paper.

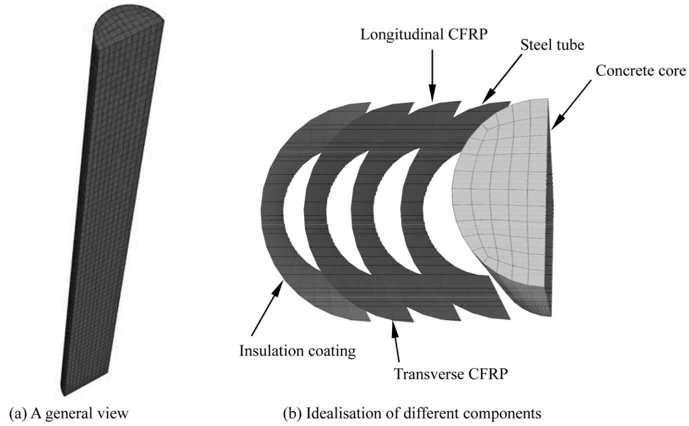


Fig 9 FE model for strengthened columns

The material thermal properties, including the thermal conductivity, specific heat and density, were required in conducting the heat transfer analysis. Those values for the concrete and steel materials reported by Lie (1994) were used in this paper, which had been used extensively in analysing temperature distributions for both CFST and reinforced concrete columns (Han *et al*. 2003, 2006). The aforementioned thermal properties for the coating material, and those varied with temperature for the CFRP which were suggested by Griffis *et al*. (1981) were used in the heat transfer analysis. It should be noted that the fire protection material was vermiculite-based. Although vermiculite is chemically stable and can be assumed to have constant thermal properties, the existence of small amounts of other substances in the material, like fire-resistant adhesive, chemical catalyst, and durability enhancing additive, may make the thermal properties of the fire protection material somewhat temperature-dependant. Unfortunately, no test data were available to determine its thermal properties as a function of temperature. Therefore, constant thermal conductivity, specific heat and density were used for the insulation material in the heat transfer analysis.

Fig. 8 provides a comparison of the temperatures predicted by the FE modelling against those measured at various points within the cross section. As can be seen, the agreement is generally good. A further comparison shows that the predicted difference between the temperatures at the inner and outer surfaces of the steel tube is within 2^{o}C during the fire exposure. This is attributed to the fact that the

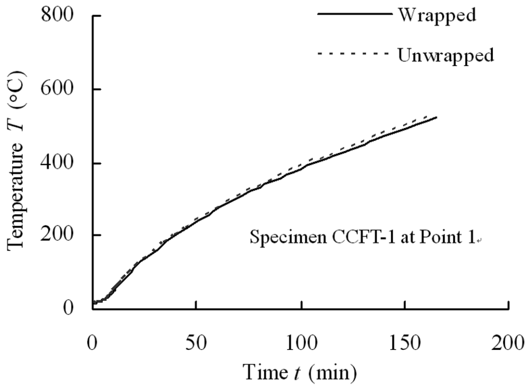


Fig. 10 Effect of CFRP wrapping on heat transfer

steel tubes used were rather thin and steel is a good conductor of heat.

Specimen CCFT-1 is selected to illustrate the influence of CFRP wrapping on the heat transfer, as shown in Fig. 10. The predicted temperature difference at the end of the test is only 8.5°C at Point 1 after taking the CFRP contribution into account. This demonstrates that the existence of the CFRP wrapping has only moderate influence on the temperature distribution across the composite section. Therefore, a little higher predicted temperatures are obtained for this specimen after ignoring the CFRP contribution, and the maximum prediction error is only about 1.6% in this case. For this reason, it is suggested by some researchers to ignore the effect of FRP on heat transfer since FRP wrapping is typically very thin (Han *et al*. 2006). However, if prefabricated composites are used in strengthening a column, the FRP jacket may be much thicker. In this case, FRP may affect the heat transfer more greatly and cannot be ignored.

The numerically calculated influence of the insulation coating on the heat transfer for the specimen CCFT-1 is shown in Fig. 11. As can be seen from this figure, the coating was still very effective in protecting the column under fire even though the applied coating was thin. Compared with an unprotected column, the temperature increase rate at any location within the protected column is much slower.

As mentioned above, the glass transition temperature (T_g) is generally not very high for normal polymer resin. According to Bisby (2005a), T_g for most polymer matrices ranges from 65°C to 150°C. Since no value of T_g was provided by the manufacturer for the resin used in this experiment, a typical value of 100°C is therefore presumed for a purpose of illustration. Fig. 12 shows the effect of the insulation thickness on the surface temperature of the FRP wrapping for the specimen CCFT-1. The FE calculation demonstrates the fact that the insulation thickness for this specimen should be at least 46 mm to keep

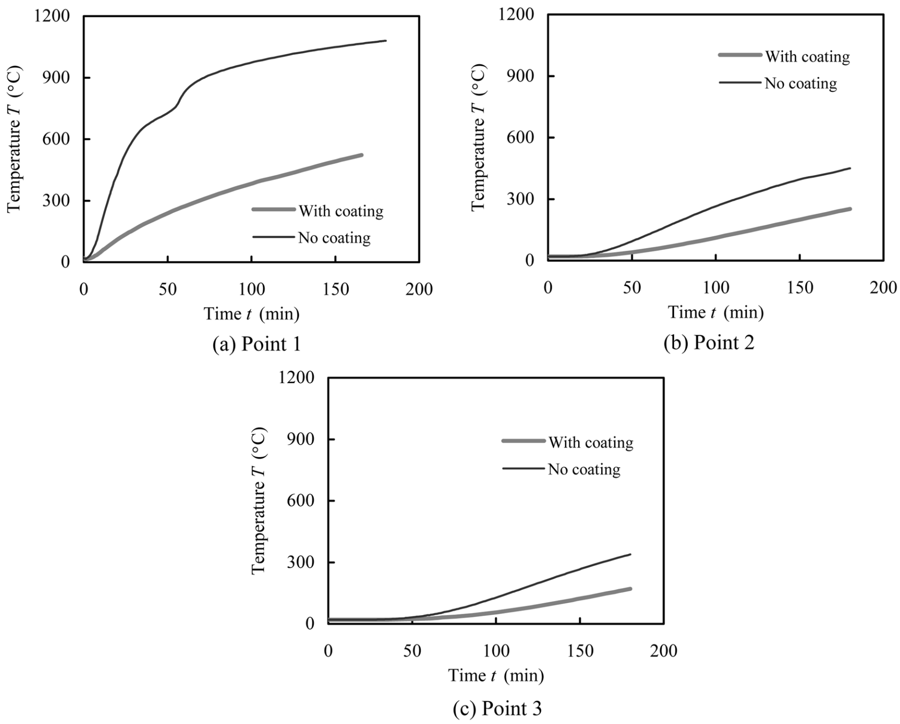


Fig. 11 Effect of insulation coating on heat transfer (CCFT-1)

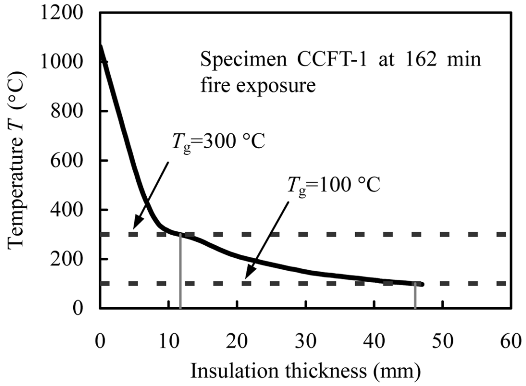


Fig. 12 Effect of insulation thickness on surface temperature of FRP wrapping

the temperature at the FRP-insulation coating interface not exceeding the presumed T_g at 162 min fire exposure. Obviously, it is not economical to use such a thick coating to protect the structural integrity of CFRP material with a low T_g value. However, with increasing efforts in enhancing the glass transition temperature of FRP composites, it has been reported that the glass transition temperature for carbon fibre reinforced composites comprised of thermosetting polyimides can reach as high as 440°C (Hao *et al.* 2001). If the heat-resistant FRP materials are commercially available and economical to be used in civil engineering in the future, it seems the required insulation thickness can be greatly reduced to protect the FRP during fire exposure. This can be seen from Fig. 12, where the required insulation thickness will reduce to 11.7 mm if T_g increases to 300°C. If FRP composites with a high T_g available easily in the future, it is likely that the FRP protection concept may be applied to reduce further repair requirement in the case of fire incidents.

3.3. Axial deformation

The variation in axial deformation Δ with fire exposure time t is shown in Fig. 13 for both test columns. As can be seen, both columns displayed a slight elongation initially, and contracted eventually. The elongation was induced as a consequence of thermal expansion. It should be noted that the

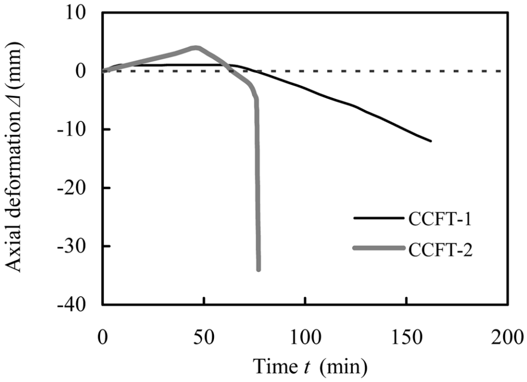


Fig. 13 Axial deformation as a function of time

magnitude of the expansion was greater for the specimen CCFT-2, which carried a smaller axial load. Thus, the influence of thermal creep was less pronounced for this specimen. Clearly, the contraction of the specimen CCFT-1 was mainly due to the thermal creep and the loss of strength and stiffness of the materials as the internal temperature increased. Since the maximum recorded temperature was only 291°C for the specimen CCFT-2 at point 1 as shown in Fig. 8(b), the deterioration of mechanical properties for both the concrete and steel was not structurally significant. Its final contraction was mainly attributed to the bending effect as shown in Fig. 7(a).

3.4. Fire resistance

The specimen CCFT-2 had a fire resistance of 77 min, whereas the specimen CCFT-1 had a fire resistance of greater than 162 min. As can be seen, the fire resistance of the axially loaded CCFT-1 was more than two times that of CCFT-2. Apart from the influence of the load eccentricity, another reason is that, under service load, the longitudinal fibres of the CFRP in an eccentrically loaded column could participate in resisting the bending moment more effectively. The load carried by the CFRP was redistributed to the steel and concrete components after the CFRP failed in carrying load, thus inducing earlier failure of the specimen. Based on FE analysis described in the following, it is found that the axial load ratio of CCFT-2 increased from 0.514 to 0.608 after the loss of the effectiveness of the CFRP wrap during fire. However, for the specimen CCFT-1, less axial load was carried by the CFRP, where the axial load ratio increased only slightly from 0.485 to 0.494 after the CFRP wrap failure.

A FE model using ABAQUS software has been developed by Lu *et al.* (2009) to predict the fire performance of CFST stub columns subjected to standard fire. Close agreement is shown between the test and predicted results in terms of the fire resistance and the axial deformation-fire exposure time curves. Similar models were also developed by Park *et al.* (2008), Hong and Varma (2009) recently.

To simulate the fire performance of CFRP-strengthened CFST columns, mechanical properties for the steel, concrete and CFRP materials, which are temperature-dependent, should be provided to conduct the stress analysis. In Lu *et al.* (2009), a classic metal material model in ABAQUS was chosen to describe the behaviour of the steel at multiple stresses state, and the concrete damaged plasticity model in ABAQUS was used for concrete. Meanwhile, the uni-axial stress-strain relationship for steel at elevated temperatures from Lie (1994) was adopted and a uni-axial stress-strain for concrete under compression was presented by Lu *et al.* (2009). The influence of thermal creep was not considered in the FE modelling directly since it had been implicitly incorporated in the material models (Lu *et al.* 2009). In the current simulation, the same models used by Lu *et al.* (2009) to describe the mechanical properties of steel and concrete were also adopted in this paper. All details of the models can be found in Lie (1994) and Lu *et al.* (2009). The FE modelling technique presented by Lu *et al.* (2009) was further developed by the authors to consider the CFRP contributions in this paper. By doing this, the fire performance of CFRP strengthened CFST columns can be simulated. It should be noted that semi-empirical relationships suggested by Bisby *et al.* (2005a) in describing the deterioration of strength and stiffness of CFRP with increasing temperature were used in the FE modelling.

The predicted axial deformation versus fire exposure time curves are compared with the test curves in Fig. 14, and the predicted fire endurance are presented in Table 1. Reasonable agreement is obtained between the predicted curves and the test results. To further compare the performance of CFSTs with and without FRP jackets, the predicted curves are also shown in Fig. 14 for unstrengthened CFST columns, which have the same parameters as those shown in Table 1 except the existence of the FRP jackets. Obviously, a strengthened CFST column has almost a same fire resistance as its unstrengthened

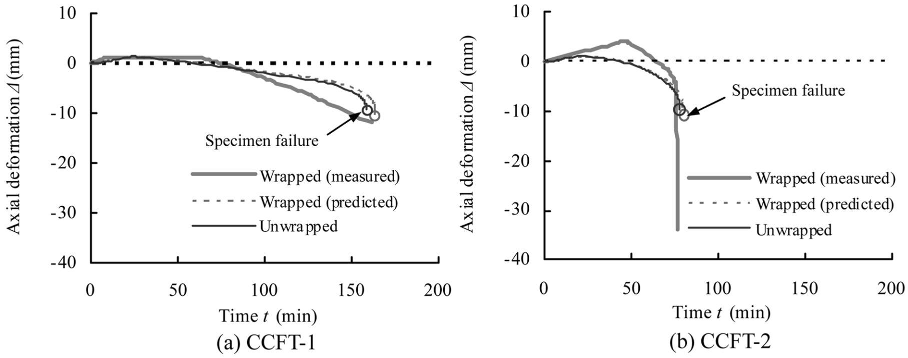


Fig. 14 Effect of CFRP wrapping on fire performance

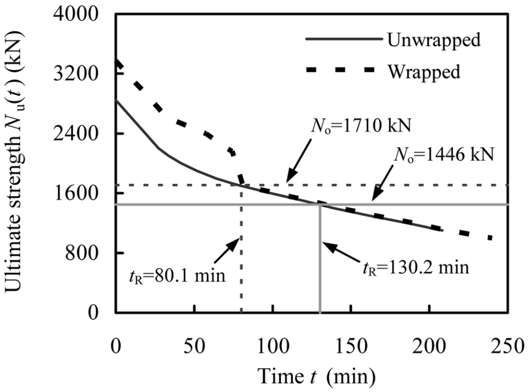


Fig. 15 Predicted ultimate strength as a function of fire exposure (CCFT-2)

counterpart without the FRP contribution. Only about 3% increase in fire resistance is achieved for the strengthened columns when taking the FRP contribution into account. This contribution is mainly resulted from the minor effect of FRP on heat transfer rather than from direct strength contribution.

The variation in predicted ultimate strength $N_u(t)$ with fire exposure time t is shown in Fig. 15 for the specimen CCFT-2. The curve for its corresponding unstrengthened column is also shown in this figure. It is clear that the load-carrying capacity of both columns decreases with increasing fire exposure time. Owing to the presence of CFRP, the wrapped column has higher initial strength than the unwrapped counterpart. But after a predicted 74 min fire exposure, the effectiveness of the CFRP strengthening system begins to deteriorate quickly. Hereafter, both columns exhibit almost identical strength. It should be noted that the predicted deterioration of the CFRP effectiveness is later than that observed in the test indicated by the combustion of the resin. This is owing to the influence of actual cracking appeared on the coating which has been ignored in the FE modelling.

If a same axial load ratio of 0.514 is applied to both the columns presented in Fig. 15, the axial load N_o applied on the unwrapped one will be 1446 kN and that on the strengthened column will be 1,710 kN due to increased service load. With a same insulation thickness, the unwrapped column is predicted to fail at 130.2 min while the calculated fire resistance t_R for the CFRP-wrapped one decreases to only 80.1 min. It is clearly that thicker insulation coating should be applied to protect strengthened CFST

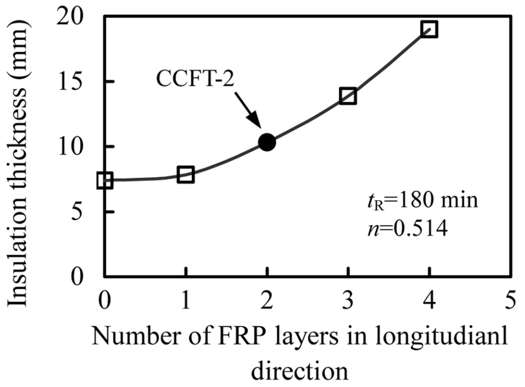


Fig. 16 Predicted ultimate strength as a function of fire exposure

columns after the structural effectiveness of the FRP system is lost during fire exposure.

Fig. 16 depicts the required insulation thickness for different strengthened columns with a same axial load ratio to achieve a target fire resistance of 3 hours. These columns have the same parameters except the applied number of FRP layers in longitudinal direction. For each column, the number of FRP sheets in hoop direction is exactly the same as that in longitudinal direction. As can be seen, the required insulation thickness increases dramatically as the number of FRP layers increases. This can be explained by the fact that the more number of CFRP layers, the higher the load-carrying capacity of a strengthened column has when the axial load ratio is kept constant. Therefore, the actual load carried by a column will increase as the number of CFRP layers increases. To obtain a same fire rating, thicker insulation will be required for a column with more CFRP layers to account for the fact that higher load will be carried by the CFST column if the strength of the CFRP deteriorates in a fire. For the test specimen CCFT-2 with two layers of CFRP in longitudinal direction, the required insulation thickness is 10.3 mm, while that for the column with two times the number of CFRP layers used is 19.0 mm. Currently, the required fire endurance in Chinese Standard GB 50016-2006 (2006) for a structural column in a building structure is 2, 2.5 or 3 hours, respectively, according to its fire resistance rating. It can be concluded that the fire endurance requirements on strengthened composite columns can be met if they are appropriately designed.

Based on aforementioned discussion, it is conceivable that it is not economical to apply a very thick insulation coating to protect the FRP materials, since the glass transition temperature for common FRP materials is comparatively low (Kodur *et al.* 2007, ACI 2008). With the infill of concrete, however, the fire resistance of CFST columns is greatly enhanced when compared with steel columns. Therefore, the loss of FRP effectiveness does not represent failure of the strengthened CFST columns. Although a thin layer of coating can not keep the FRP intact, it can greatly improve the fire resistance of the CFST columns since the insulation system can delay strength degradation of the steel and concrete under fire exposure. Therefore, in designing FRP-strengthened CFST columns with a thin insulation coating, the FRP strengthening effect may be considered to be effective only at ambient temperatures, and the fire resistance of the strengthened columns can be determined by that of CFST columns, assuming that the strength of externally bonded FRP system is lost completely in a fire. Similar design guideline was put forward in ACI 440.2R-08 (2008) for the fire design of FRP-strengthened concrete columns.

However, the above design guidelines will be conservative when a thicker insulation coating is applied to protect the FRP jacket. According to Bisby (2005a), the residual tensile strength of CFRP at a temperature of 400°C is around 30% of that at ambient temperatures. When the temperature becomes even higher, the strength of CFRP can be ignored. It can be found from Fig. 12 that the surface temperature of FRP jacket is likely to be lower than 400°C when the insulation thickness (a) is larger than 10 mm and the fire exposure time is less than 180 min. Therefore, it is recommended that the residual strength of CFRP may be considered when $a \geq 10$ mm. In this case, more economical, thinner insulation can be applied to achieve the required fire rating.

It should be pointed out that, even when the insulation thickness (a) is less than 10 mm, partial strengthening effect for a FRP-strengthened column may still be considered in an actual fire rather than the standard fire, since the FRP temperature may remain low if using a fire performance based design approach. Further research is needed to address this issue.

4. Conclusions

This paper is an attempt to study the fire performance of concrete-filled steel tubular columns strengthened by CFRP. From two tests conducted, it was observed that the CFRP material covered by a thin insulation coating was susceptible to combustion in fire. This coating, however, could effectively delay the failure of the composite columns. Although a thin layer of coating can not keep the CFRP intact, it can greatly improve the fire resistance of the CFST columns. The fire resistance of the CFRP-strengthened CFST columns can be enhanced through the use of conventional fire-protection coat for steel structures. In designing CFRP-strengthened CFST columns, if the insulation thickness is less than 10 mm, strength of externally bonded CFRP system can be assumed to be lost completely in a fire, because of the degradation of most FRP materials at high temperatures. Otherwise, the residual strength of CFRP may be considered.

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