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Fire performance of concrete-encased CFST columns and beam-column joints

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Abstract

Concrete-encased CFST (concrete filled steel tube) structure is a type of composite structure featuring an inner CFST component and an outer reinforced concrete (RC) component. They are gaining popularity in high-rise buildings and large-span buildings in China nowadays. To date, the behaviour of concrete-encased CFST structures at ambient temperature has been investigated, but their fire performance has seldom been addressed, including the performance in fire and after exposure to fire. This paper summarizes the fire test results of concrete-encased CFST columns and beam-column joints. The cruciform beam-column joint was composed of one continuous concrete-encased CFST column and two cantilevered reinforced concrete (RC) beams. These specimens were subjected to a combined effect of load and full-range fire. The test procedure included four phases, i.e. a loading phase at ambient temperature, a standard fire exposure phase with constant load applied, a sequential cooling phase and a postfire loading phase. The main findings are presented and analysed. Two types of failure were identified, i.e. the failure during fire exposure and the failure during postfire loading. Global buckling failure was observed for all the column specimens. The column specimens with common load ratios achieved high fire ratings without additional fire protection. The concrete-encased CFST columns also retained high postfire residual strength. As for the joint members, beam failure was observed in all cases. The measured temperature-time history and deformation-time history are also presented and discussed. For both the column and joint specimens, the deformation over the cooling phase was significantly greater than that in the standard fire exposure phase.

Keywords: *Concrete-encased; Concrete filled steel tube (CFST); Reinforced concrete (RC); Composite columns; Fire resistance; Postfire residual strength.*

1. Introduction

Concrete-encased CFST (concrete-filled steel tube) is a type of composite structure, whose cross section is composed of an inner CFST component and an outer reinforced concrete (RC) component. Fig. 1(a) shows one typical cross section, which is made up of an inner circular CFST component and an outer RC component as an encasement. This kind of composite members was initially used in high-rise buildings in China in the 1990s to attain high seismic performance. So far, they have also been widely used in large-span buildings, bridges, and subway stations. In construction practice, favourable seismic behaviour can be achieved by (1) using concrete of higher strength grades for the inside of the steel tube than the outside, and (2) casting the inner concrete first and employing the inner CFST component to sustain the construction loads afterwards. On this condition, the inner CFST component has greater load ratio in service stage than the outer RC component, which allows the outer RC component, especially the outermost concrete fibre, to obtain more capacity of developing compressive stress when subjected to seismic action. The static performance and seismic performance have been well addressed in literatures of [1]–[4] and [6]–[9], respectively.

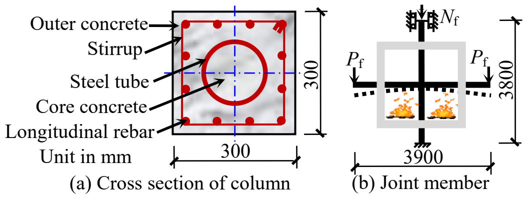


Fig. 1. Dimensions of concrete-encased CFST specimens.

Concrete-encased CFST members are also expected to achieve good fire resistance (both during fire exposure and after fire) due to the insulating effect of the outer RC component. However, the fire performance of concrete-encased CFST members has seldom been addressed. But the fire performance of other composite columns has been well addressed. Extensive research is available on the fire performance of CFST column, such as [10]–[20]. Recently, Neuenschwander et al. [21] presented an investigation into the fire behaviour of CFST column with solid steel core. The results have confirmed the design concept that redistribution of the load sharing occur in the fire situation, and the steel core bear the load initially bore by the outer part. Similarly, concrete-encased CFST might also achieve the redistribution of load sharing effect. Additionally, the behaviour of seismic affected concrete-filled double skin tube column has been investigated [22]. These researches provide a basis for the investigation into the fire performance of concrete-encased CSFT structures.

Set against this background, experimental and numerical investigations on the fire performance of concrete-encased CFST columns and beam-column joints were conducted by Zhou [23]. The purposes of these tests were: (1) to create a basic database of the fire tests of concrete-encased CFST columns and beam-column joints and (2) to provide test evidence for further numerical investigations and (3) to lead to code coverage to facilitate the practical fire design and postfire assessment of this kind of structures. This paper summarizes the test programs and presents the main findings through tests.

2. Fire performance of concrete-encased CFST columns

Fire performance includes the performance during fire exposure and that at ambient temperature after exposure to fire. Concrete-encased CFST columns are expected to achieve high fire ratings without additional fire protection because the inherent layout of the cross-section, in which the inner CFST component is insulated by the outer RC component. The inner CFST component could achieve a temperature much lower than the outer component in the fire situation due to the high thermal inertia of the outer concrete. This allows the occurrence of redistributions of load share within the cross-section when the concrete-encased CFST columns are subjected to fire exposure.

Experimental investigations of the performance of concrete-encased CFST members in fire and after exposure to fire have been conducted [24]. The members include column specimens and beam-column joints. The postfire tests were conducted following the temperature-load-time path as shown in Fig. 2. The specimens were initially loaded at ambient temperature (AA'). And then they were exposed to fire including a standard fire exposure phase (A'B') and a natural cooling down phase (B'C'), during which the load was kept constant. After the specimens returned to ambient temperature (C'D'), the load was increased until the specimen failed (D'E').

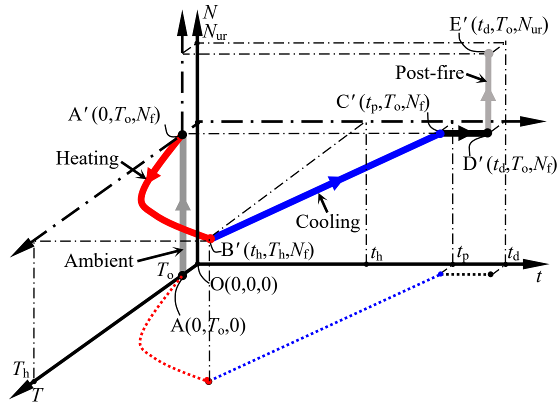


Fig. 2. Temperature (T)-load (N)-time (t) path.

2.1. Composite columns in fire

Six fire resistance tests of concrete-encased CFST columns were conducted. Normal weight concrete was used inside the steel tube, while fine stone concrete was used outside the tube. The 28-days concrete cube strengths for normal weight concrete and fine stone concrete were 56.4 MPa and 24.9 MPa respectively, and their cube strengths at the time of test were 60.1 MPa and 31.2 MPa respectively. Two type of steel tubes were used, and their outer diameters were

159 mm and 203 mm, respectively. Their corresponding yield strengths were 416 MPa and 398 MPa respectively, and the ultimate strengths were 642 MPa and 555 MPa respectively. The longitudinal rebars were 16 mm in diameter with a yield strength of 363 MPa and an ultimate strength of 558 MPa. The test parameters included load ratio (defined by the ratio of the load applied in test to the ultimate strength at ambient temperature), diameter of the inner steel tube (or the steel tube ratio) and heating time ratio (defined by the ratio of the target standard fire exposure time in test to the fire resistance obtained in the reference fire resistance test). The axial displacement-time history, rotation of the upper endplate and the temperature-time history were recorded. The failure criteria proposed by ISO834-1[25] was used.

The test results confirmed that concrete-encased CSFT columns can achieve high fire ratings without additional fire protection. The specimens with common load ratio of 0.42 achieved fire resistance of more than 160 min. The specimens with low load ratio of 0.30 had fire resistance of 201 min.

Fig. 3 shows the failure mode of one fire resistance test. The load ratio applied during fire exposure was 0.3. The fire resistance of this specimen was 201 min. It can be found that global buckling was observed for this test. The residual deformation curved towards the west, with the peak deflection appeared at mid-height. This residual deformation confirmed that the fixed-fixed boundary conditions were well maintained during the test, although the measured displacement from the four LVDTs on top of the specimens revealed that a finite rotation occurred in the top end. Even though concrete corner spalling occurred, no evident concrete explosive spalling could be observed. That is mainly because the strength of outer concrete was 31.2 MPa, which was less possible to cause explosive spalling. The corner spalling of the concrete exposed the inner reinforcements, whose surface was blackened by fire exposure. A wide longitudinal crack was observed in the north side of this specimen. A close visual observation revealed that this wide crack was formed by the connection of a large number of inclined minute cracks. These cracks formed along with the bending deformation of the specimen and were caused by the shear stress on the surface of the outer concrete.

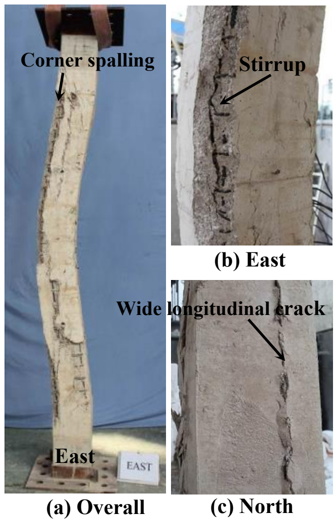


Fig. 3. Failure mode of fire resistance tests.

The measured axial displacement (Δ_c) versus time (t) relationship of a fire resistance test is shown in Fig. 4. The Δ_c-t relationship is characterized by three stages, i.e. an expansion stage (OA), a compression stage (AB) and a failure stage (BC). The expansion of this specimen peaked at 109 min with maximum expansion of 3.75 mm. After keeping compressing in the following 93 min, the specimen finally failed at 201 min. It shows that concrete-encased CFST column failed in a ductile way during fire exposure.

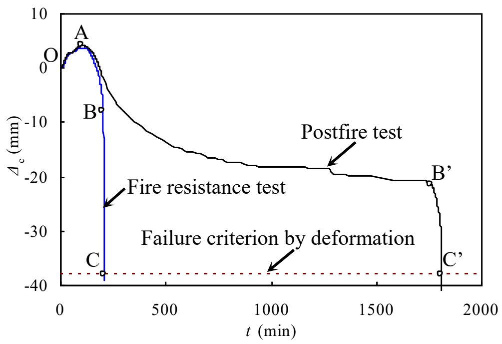


Fig. 4. Axial displacement (Δ_c) versus time (t) relationships.

2.2. Composite columns after exposure to fire

Postfire tests of concrete-encased CFST columns were also conducted after the fire resistance tests [23]. Detailed information can be found in [24] and [26]. The reason for conducting postfire tests was that the concrete-encased CFST column may probably survive short-team fire exposure due to the insulating effect of the outer RC component, and postfire assessment would be another issue that needs to be addressed.

In all, four postfire full-scale tests were conducted. The fire exposure was determined according to the corresponding fire resistance tests with identical test parameters. Heating time ratios of 0.33 or 0.67 were used to determine the target fire exposure time. The loading at ambient temperature and the standard fire exposure phase were identical to that of fire resistance tests. After the target fire exposure was attained, the furnace was switched off. The specimens were cooled down naturally with the furnace fans kept running until the temperature readings of all the furnace thermocouples dropped below 100ºC. The furnace was then opened to further cool down the specimens. The load was increased step by step until the specimen failed. The temperature-time history over the full-range fire, the displacement and force time-history were recorded.

Fig. 5 shows the failure mode of one typical postfire tests. The load ratio was 0.3, and the heating time ratio was 0.67 (fire exposure time was 137 min). Although this specimen was subjected to 137 min of fire exposure, neither fire-induced concrete explosive spalling nor wide surface cracks were observed after fire exposure, as shown in Fig. 5(a). Evident plastics hinges formed slightly lower than mid-height of the column [Fig. 5(b)]. The concrete cover was crushed in the compressive zone, and the inner longitudinal rebars buckled outwards [Fig. 5(c)]. It is noticeable that water trace existed below the lower vent hole after fire exposure, as shown in Fig. 5(d). It indicated that the existence of the vent holes was essential.

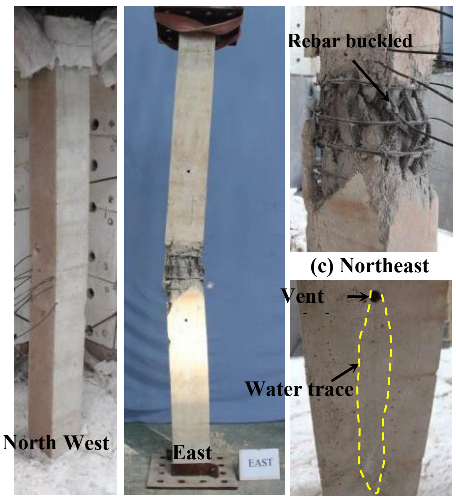


Fig. 5. Failure mode of fire postfire tests.

Fig. 6 shows the measured specimen temperature (T) versus time (t) relationships. For clarity, the only the temperature of the first 500 min are displayed herein. It can be found that: (1) Plateau stage around 100ºC near steel tube are more obviously observed than the other parts. The plateau stages around 100ºC of points 2, 3, 6 and 7 last longer than the other points. It is because moisture contained in the concrete near the tube need to migrate a longer distance to the outer air compared to the outer part, while this part is also vulnerable to high temperature during fire exposure. (2) Temperature differences were captured between the inner and outer tube surfaces. That could be resulted from possible measurement error existing at the measuring end of the thermocouples whose diameter was 6 mm. Furthermore, the formation of the gap in the interface of the concrete and steel tube might also provide a plausible excuse for this difference. (3) The temperature attained by inner CFST component was low due to the insulating effect of the outer RC component. Even though the fire exposure was 137 min, the maximum temperature attained by the steel tube was lower than 500ºC. It indicates that the properties of steel were seldom degraded by the fire exposure [27], which contributed to the high residual strength retained after fire as well as the ease of repair.

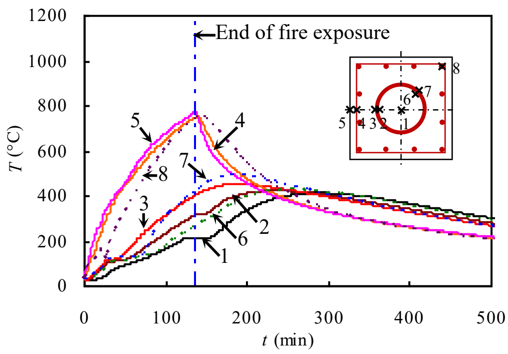


Fig. 6. Measured temperature (T) versus time (t) relationships of postfire test.

The measured axial displacement (Δ_c) versus time (t) relationship of a postfire test is shown in Fig. 4. Similarly, this relationship is characterized by three stages. The peak expansion of 4.20 mm was attained at 103 min of fire exposure, which is close to that of the corresponding fire resistance test. The axial displacement changed by 25.56 mm in the compression stage (AB'). It should be noted that this specimen had already started to compress before the fire exposure ended. The Δ_c-value at 137 min was 3.53 mm. It can be found that the compression during cooling phase was 24.89 mm, which was over 37 times greater than that occurred during the fire exposure phase. This indicates that the influence of cooling down phase should be considered when numerically calculating the deformation of concrete-encased CFST columns after fire.

3. Fire performance of concrete-encased CFST column to beam joints

The inherent layout of the cross-section of concrete-encased CFST column makes it adaptable to be connected to both RC beam and steel beam. The former was selected as the research object and its fire performance was experimentally investigated [23]. This section summarises the fire test results of concrete-encased CFST column to RC beam joints.

3.2. Composite joints in fire

The fire resistance tests of the concrete-encased CFST column to RC beam joints were conducted by Zhou [23]. Fig.1(b) shows the dimensions of the joint members. These joints were designed based on the above concrete-encased CFST columns and current Chinese design codes. The steel tube and the longitudinal rebars penetrated the joint zone to maintain the continuity. The RC beam bulged in the joint so that the longitudinal rebars can bypass the steel tube.

The adopted test procedure was similar to the column tests. Both the column top and the two beam ends were loaded vertically. During the standard fire exposure phase, the loads were kept constant. The bottom surfaces of the RC slab and RC beam, two sides of the RC beam and the four sides of the lower half of the column were exposed to fire. Failure was deemed to have occurred if either the column or the beam failed according to ISO 834-1 [25].

Fig. 7 shows the failure mode of a fire resistance test, with the views of the slab top surfaces and beam bottom shown in the corner subfigures. The load ratios of column and beam were 0.30 and 0.28, respectively. The test was terminated at 208 min, and the fire resistance was attained at 200 min.

Beam failure was observed in this test, and the beam showed a smooth curvature of residual bending deformation throughout the fire-exposed length. Cracks on the top surface of the RC slab can be observed, both on the north and south sides. The crack with maximum width appeared in the midspan of both sides of the cantilever slab. By comparison, the concrete at the bottom of RC beam crushed slightly, with no inner reinforcement exposed. And the bottom surfaces of the RC slab showed no evidence of cracks. Slight bending deformation can be detected visually in the column just below the joint zone. No fire-induced concrete explosive spalling was observed. The concrete corner spalling was observed around the mid-height of the lower half of the column.

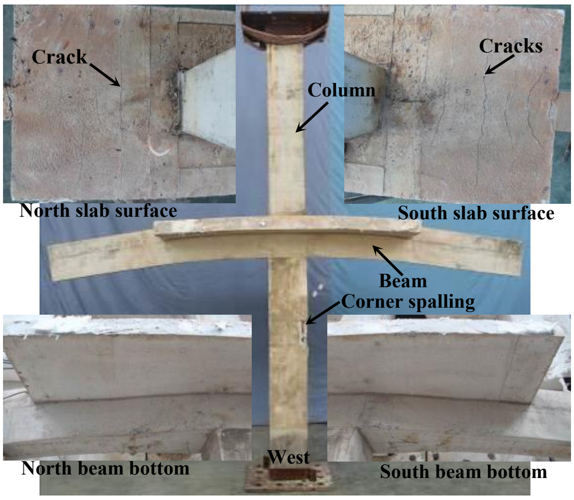
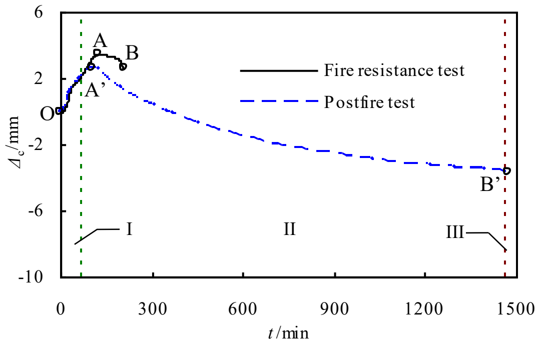


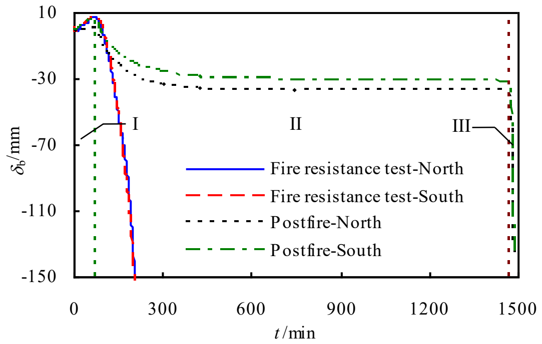
Fig. 7. Failure mode of fire resistance test.

The column axial displacement (Δ_c) versus time (t) relationships are shown in Fig. 8(a), in which expansion is positive and compression is negative. This Δ_c-t relationship can be divided into two stages, i.e. an expansion stage (OA) and a compression stage (AB). The peak expansion was 3.47 mm attained at 125 min. Compared with the Δ_c-t relationship of column member in Fig. 4, it can be found that the column in joint member attained a less peak expansion. That is because only the lower half of the column was exposed to fire in joint member.

The beam deflection (δ_b) versus time (t) relationships are shown in Fig. 8(b). The deflection is positive up and negative down. The beam ends deformed upwardly, because the effect of thermal elongation of the beam bottom outweighed the deflection caused by the loads. The north beam attained the peak deformation of 7.58 mm at 74 min, while the south beam attained the peak deformationn of 7.68 mm at 69 min. The difference was mainly caused by the imperfections of the specimen.



(a) Column axial displacement (Δ_c) versus time (t) relationships



(b) Beam deflection (δ_b) versus time (t) relationships

Fig. 8. Deformation versus time relationships.

3.3. Composite joints after exposure to fire

The postfire tests of concrete-encased CFST column to RC beam joints were performed after the fire resistance. The fire exposure time was determined according to the corresponding fire resistance test. The test procedure was similar to that of the column postfire test. The load on the column and that on the ends of the beams were kept constant during the full-range fire. In the postfire phase, the loads on both beam ends were increased step by step until the specimen failed, while the load on top of the column was kept constant. The beams were deemed to have failed if the increase of the load could not be maintained. The load and displacement time-history were recorded in the postfire loading phase.

The failure mode of a postfire test is shown in Fig. 9, with the views of the top surfaces of the slab and the bottom views of the beam shown in the four corner subfigures. The load ratios of column and beam of this specimen were 0.30 and 0.25, respectively. The fire exposure was 0.33t_R (=72 min), in which t_R was the fire resistance of the corresponding fire resistance test.

Beam failure was observed for this postfire test. The residual strengths of both beams were 60.0kN. Wide cracks were observed at the top surface close to both supported ends of the cantilever slabs. The cracks penetrated the RC slab, and even extended to the upper half of the RC beams, as observed in the views of the bottom of the beams. By comparison, the wide cracks in fire resistance tests concentrated in the mid-span of the RC slab (as shown in Fig. 7). This is because the postfire specimens were tested after the specimens returned to ambient temperature, and the bending moment peaked at the joint end. But for the fire resistance specimens which were tested at elevated temperature, the joint zone was a cooler end due its heat sink effect. The column in this joint member showed no observable lateral deformation around the mid-height zone, which differed with the case of the fire resistance test.

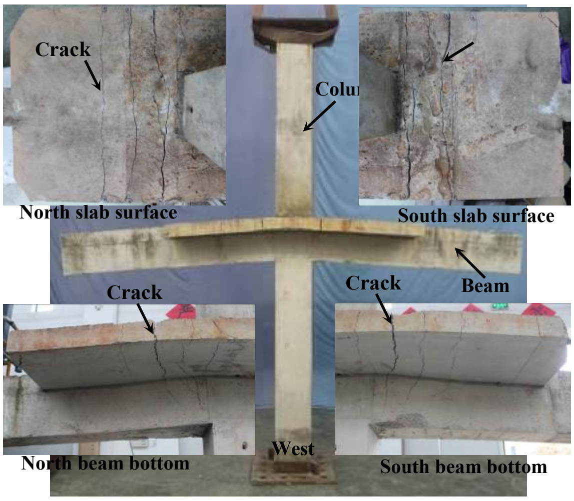


Fig. 9. Failure mode of postfire test.

The deformation versus time relationships of the postfire test are shown in Fig. 8, in which I denotes the loading phase at ambient temperature, II denotes the standard fire exposure phase and cooling down phase, and III denotes the postfire loading phase. The Δ_c-t relationship of postfire test can also be divided into three stages, which are similar to those of the column specimens in Fig. 4. The column in the joint member kept compressing over the cooling down phase, which lasted for over 25 h, as shown in Fig. 8(a). Fig. 8(b) shows the δ_b-t relationship of postfire test. It can be found that the deflections developed obviously in the initial 400 min of cooling phase, while they kept stable after 400 min. The deflection of the north beam exceeded that of the south beam due to possible imperfections and load eccentricity. The deflection of the north beam increased by 0.37 mm during fire exposure after the peak positive deflection was attained, while it developed by 36.18 mm (more than 97 times than the former) in the cooling phase. This again confirms that cooling phase has tremendous effects on the deformation of structural members after fire.

4. Concluding remarks

This paper summarizes the results of fire tests of concrete-encased CFST columns and beam-column joints. The cross sections of the concrete-encased CFST column used were square with inner circular steel tube. The test parameters were load ratio, diameter of steel tube and fire exposure time. This paper presents the main test observations and analyses the test results. Four typical specimens were selected as representatives, i.e. one fire resistance test and one postfire test for both columns and joints.

The test results have confirmed that concrete-encased CFST column with common load ratio can achieve high fire ratings without additional fire protection and retain high residual strength after fire exposure. Global buckling failure mode was observed for all the column specimens, and beam failure mode was observed for all the beam-column joints. The joint tests demonstrated that the concrete-encased CFST column with common load ratio would not dominate the failure of joint. For both the column and joint specimens, the deformation that occurred during cooling phase was significantly greater than that during fire exposure.

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