Fire resistance of steel beam to square CFST column composite joints using RC slabs: Experiments and numerical studies

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Abstract: In this paper, experimental investigation and numerical simulation of steel beam to square concrete-filled steel tube (CFST) column composite joints that use reinforced concrete (RC) slabs subjected to localized and global fire conditions are presented. Eight joints were tested under the ISO 834 fire standard, and the effect of different parameters including the load ratio of beams, the beam-to-column ratio of linear stiffness, and different fire scenarios was studied during testing. The failure patterns and the thermal responses of the structural members including the temperature distribution, axial displacement of columns, vertical deflection of the beam ends, and fire resistance of the joints were recorded and discussed. The results show that tube buckling of the square CFST columns, flange buckling of the steel beams, and separation between the top flange of the steel beams and the RC slabs were the primary failure patterns of this type of joint. Moreover, the temperatures of structural members within the connection zone were lower than those in the other regions. Compared with other factors, the load ratio of the beams demonstrated a significant influence on the displacement of the structural members and the fire resistance of the joints. A three-dimensional finite element analysis (FEA) model was built to simulate the fire performance of this type of composite joint. The simulation results were compared to the test results in terms of failure patterns, temperature distributions, displacements, and fire resistances, and good agreement in general was achieved. Finally, the FEA model was adopted to examine the effect of parameters on the fire resistance of the composite joints with axial and flexural constraints applied at the ends of the beam.

Key words: square CFST column; steel beam; composite joint; RC slab; fire performance; fire resistance; experiments; FEA simulation; localized fire; global fire

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1. Introduction

Concrete-filled steel tube (CFST) columns exhibit excellent merit over bare steel or reinforced concrete (RC) columns because of the composite action between the core concrete and the steel tube (Han et al. 2014) [1], giving rise to favourable properties such as increased bearing capacity, good plasticity and toughness, and their potential for fast construction. Furthermore, CFST members demonstrate better fire resistance than those with hollow steel tubes, as the concrete infill can delay the temperature increase in the outer steel tube by helping to absorb the heat, and the composite action between core concrete and its steel tube can be partially maintained under high temperatures (Yang et al. 2008; Rush et al. 2012; Han et al. 2018; Song et al. 2018; Zhou and Han 2018) [2-6], which results in the better performance of CFST structures than hollow steel structures in terms of fire-resistant design.

Many experimental and numerical studies have been conducted on CFST columns under fire conditions, and different simplified methods for measuring the fire resistance of CFST columns have been proposed, as summarized in Song et al. (2018) [5] and Espinos et al. (2012) [7]. However, the fire resistance of a single CFST column is evidently different from its performance as part of a composite structure, due to the variation in its loading status under the effect of neighbouring members. Therefore, it is necessary to investigate the overall structural behaviour of CFST structures in fire, considering the fact that the fire resistance performance of CFST structures to a large extent depends on the thermal responses of the beam-to-column joints, which are key components of the structure. Additionally, the moment capacity and rigidity of the beam to CFST column joints change with the deterioration of the material properties under fire conditions, which may dominate the internal force distribution and fire resistance of the structure. Accordingly, it is vital to conduct a comprehensive study of the fire performance of beam to CFST column joints.

Extensive experimental studies and theoretical analysis of the behaviour of beam to CFST column joints at ambient temperatures have been conducted (Ricles et al. 2004; Han and Li 2010;

Wang et al. 2018) [8-10]. Additionally, the performance of steel beam to column joints under fire conditions have been investigated in depth, and the moment versus temperature relationship under high temperatures has been elucidated through a number of tests, theoretical analyses, and finite element simulations (Al-Jabri et al. 2008; Wang 2011) [11,12].

Despite the great focus on beam to CFST column joints at ambient temperatures, and steel beam to column joints under fire conditions, the fire performance of beam to CFST column joints is still unclear. According to an experimental study of the fire performance of eight steel beam to rectangular CFST column joints with extended end plate connections, Wang and Davies (2003) [13] found that the effective length of the CFST column in the joints was determined by the buckling of its outer steel tube, and the moment in the CFST column varied with the temperature field. Ding and Wang (2007) [14] reported that the catenary action of steel beams was produced in the joint specimens, and by conducting an experiment on the fire performance of ten steel beam to circular and square CFST column joints with different types of connections, found that joints with reverse channel connections possessed higher stiffness, strength, rotational capacity, and ductility than the others. Han et al. (2007) [15] developed a finite element analysis (FEA) model to simulate the hysteretic performance of steel beam to circular and square CFST column joints with external ring plates after fire exposure, and their FEA results were validated by experimental observation. Han et al. (2008) [16] conducted fire tests of RC beam to circular and square CFST column joints to evaluate the influence of axial load of the CFST column, load ratio of the RC beam, and beam-column stiffness ratio on the fire characteristics of the joint specimens, and their experimental results were further used to validate the FEA model developed by Tan et al. (2012) [17]. The experimental results, and the corresponding FEA model of steel beam to circular CFST column joints with RC slab subjected to simulated fire, including the cooling phase, were presented by Song et al. (2010) [18], and the FEA model was then used to predict the temperatures, deformations, and moment versus relative rotation angle relationship of composite joints under a combination of loading and fire, including the heating and cooling phases. Song and Han (2014) [19] conducted a numerical investigation into the post-fire behaviour of the constrained steel beam to circular CFST column joints under a complete loading and fire process.

The above review of the existing literature illustrates that there have been limited research achievements related to the fire performance of steel beam to square CFST column composite joints using RC slab. Compared to joints that use circular CFST columns, this type of composite joint has advantages such as greater stiffness and moment capacity of the square CFST column, and easier welding of external ring plates due to its configurations. In addition, the steel ring plates connecting square CFST column and steel beams also enable a higher moment capacity than other connections. Consequently, this form of joint configuration is preferred by structural engineers. As little research has been done on the fire performance of such joints, it is necessary to carry out new research on its fire behaviour to support fire safety design for this kind of composite joint.

The aim of this work is to conduct an experimental and numerical study of the failure patterns, temperature development at different positions, displacement versus heating time relationship, and fire resistance of the joints exposed to ISO 834 standard fire (ISO 834-1, 1999) [20]. In this study, eight joint specimens with various load ratio of beams, ratio of linear stiffness of beam to column, and different fire scenarios were tested, and the observations in the tests were adopted to further evaluate the accuracy of the fire performance simulated by the FEA model of the joints. Finally, the validated FEA model was further used to investigate the effect of different parameters on the fire resistance of the composite joints.

2. Experimental investigation under fire conditions

2.1. Test arrangement

When a planar frame structure is exposed to fire, its interior joints may be exposed to different temperatures, and may exhibit different structural responses. The present study mainly focuses on two types of situations: (1) when fire develops underneath the slabs adjacent to the interior beam to column joint, which is denoted as a localized fire, and (2) when all the structural components in the connection zone are exposed to fire, which is denoted as a global fire, as shown in Fig. 1, where

H_c and L_b are the height of the columns and the span of the beams respectively, and p is the vertical load on the beams. Considering the fact that the arrangement of the joints, boundary conditions, and stiffness are symmetrical, the actual test specimens are simplified into beam to column joints with a half-beam on each side of the connection, and upper and lower half-columns, which are subjected to localized and global fires, as shown in Fig. 2. The simplified boundary conditions and loading status of the joint were designed as follows: both the top and bottom end of the column are hinged, with an axial compressive load (N_F) applied on the top, and both ends of the beam are free with a vertical load (P_F) applied to both ends.

2.2. Specimen preparation

Each specimen consisted of a square CFST column and two H-shaped steel beam segments supporting an RC slab in cruciform arrangement to represent an interior beam to column joint in a planar frame structure. Two steel ring plates were used to connect the square CFST column and H-shaped steel beam segments, as demonstrated in Fig. 3. As the internal length of the furnace chamber was shorter than the total length of the two beam segments in the joints, two small rectangular holes, 600 mm high and 500 mm wide, were provided, at the opposite wall of the chamber to accommodate the beam length. As a result, the dimensions of the RC slab were designed according to the configurations of the furnace chamber, and there was no RC slab on the beam outside the furnace chamber. Each beam segment was divided into two parts to facilitate the installation of the joint components from the top of the chamber, and the steel studs were mounted between the RC slab and the H-shaped steel beam. Furthermore, four pairs of stiffeners were welded to the steel beam to improve the stability of web plate. The design of the H-shaped steel beam was in accordance with the Chinese standard GB 50017 (2017) [21], and the design of the square CFST columns and the steel ring plates in the connection zone were based on the Chinese specification DBJ13-51 (2003) [22]. The studs were placed in a single line along the longitudinal direction of the RC slab, and the length, diameter, and spacing of the studs were 70 mm, 7 mm, and 75 mm, respectively. Longitudinal bars with a diameter of 7.5 mm and a spacing of 140 mm, and distribution bars with a diameter of 7.5 mm and a spacing of 250 mm were arranged in the RC slab with a 30 mm cover.

The joint specimens were prepared according to the following sequence: firstly, two cold-formed U-shaped steel profiles were welded with full butt weld into a tube according to the designed dimensions, as well as other accessories, including the steel ring plates, beam segments, and stiffeners; secondly, two ring plates were welded to the square tube at the designated locations as well as to the beam segments; thirdly, the concrete was cast into the square tube after all the thermocouples were placed into pre-embedment brackets on the tube wall; finally, the formwork for the RC slab was placed on the top flange of the steel beam after the studs were welded, and the concrete was poured into the formwork after the installation of the steel bars and thermocouples. In addition, the top and bottom surfaces of the square CFST columns were grinded using a grinder to ensure a smooth surface, so the axial compressive load could be applied to the steel tube and its concrete core simultaneously. During the assembly, the axes of the left and right segment of the steel beam were ensured to coincide with each other, and the axes of two beam segments and square CFST column were kept in the same plane.

A summary of the joint specimens is presented in Table 1, where B_c and t_c are the width and thickness of the steel tube in the square CFST column, respectively; h_b, b_f, t_w, and t_f are the height, flange width, web thickness and flange thickness of the H-shaped steel beam, respectively; b_{slab}, t_{slab}, and L_{slab} are the width, thickness, and length of the RC slab, respectively; k is the ratio of linear stiffness of beam to column; n is the axial compression ratio of square CFST column; m is the load ratio of beams; and $t_{R,e}$ and $t_{R,p}$ are the experimental and predicted fire resistance, respectively.

The parameters considered in the tests include:

- Load ratio of beams (m): 0.1 and 0.35;
- Ratio of linear stiffness of beam to column (k): 0.2 and 0.3; and
- Fire scenarios: localized and global.

The load ratio of beams (m) is defined as:

$$m = P_{\mathrm{F}} / P_{\mathrm{u}} \tag{1}$$

where, P_{u} is the bearing capacity of the beam at ambient temperature determined by $M_{\mathrm{u}} / (L_{\mathrm{b}} / 2 - B_{\mathrm{c}} / 2)$, and M_{u} is the hogging moment resistance of the steel-concrete composite beam in accordance to the Chinese standard GB 50017 (2017) [21].

The ratio of linear stiffness of beam to column (k) is calculated as follows:

$$k = \frac{(EI)_{\mathrm{b}} / L_{\mathrm{b}}}{(EI)_{\mathrm{c}} / H_{\mathrm{c}}} \tag{2}$$

where $(EI)_{\mathrm{b}}$ and $(EI)_{\mathrm{c}}$ are the flexural stiffness of steel beam and square CFST column, which are respectively calculated according to the Chinese standard GB 50017 (2017) [21] and DBJ 13-51 (2003) [22]. The contribution of the RC slab to $(EI)_{\mathrm{b}}$ was not considered due to the small tensile strength of the concrete, as it was subjected to tension while the downward load was applied to the beam ends.

The axial compression ratio of the square CFST column (n) was determined by the following formula:

$$n = N_{\mathrm{F}} / N_{\mathrm{u}} \tag{3}$$

where N_{u} is the stability bearing capacity of the square CFST column at ambient temperature, which can be calculated using the formulae in the Chinese standard DBJ 13-51 (2003) [22].

To monitor the development of temperatures at typical positions, four sections were selected in which to arrange the thermocouples, as demonstrated in Fig. 3. Sections C(B)J and C(B)N represent the connection zone and the non-connection zone of the column (beam), respectively. Four thermocouples were installed on the pre-welded bracket in sections CJ and CN, and two and three thermocouples were set in the sections BJ and BN, respectively.

2.3. Material properties

Standard tensile coupon tests were conducted to obtain the material properties of steel. Three tensile coupons were cut from the wall of the steel tube in the square CFST column, the flange and web of the steel beam, and the long steel rebar. From these tests, the average thickness or diameter, yield strength (f_y), tensile strength (f_u), elastic modulus (E_s), Poisson's ratio (μ_s), and elongation after fracture (δ_{ef}) of steel were obtained and listed in Table 2. The f_y , f_u , and δ_{ef} of the steel studs provided by the manufacturer were 240 MPa, 490 MPa, and 15%, respectively.

Two types of concrete were prepared, one for the in-filled concrete in the square CFST columns, and one for the concrete of the RC slabs. The mix proportion of concrete is given in Table 3. To achieve the compressive strength and elastic modulus of concrete, a number of cubes with a side length of 150 mm and three prisms with dimensions 150 mm × 150 mm × 300 mm for each type of concrete were cast and cured in conditions similar to the joint specimens. The obtained cube compressive strength at 28 d ($f_{cu,28}$), and at the time of the test ($f_{cu,test}$), and the elastic modulus (E_c) of concrete are also presented in Table 3.

2.4. Test set-up

A furnace was used to conduct fire resistance tests of the joints, and the interior dimensions of the furnace were 3000 mm length, 3000 mm breadth, and 3300 mm height. The test set-up is displayed in Fig. 4. The fire heating was achieved by eight liquefied petroleum gas burners on two sides of the furnace chamber, and the precision of the temperatures in the furnace was ±10 ^{o}C. The furnace temperatures were measured with five chromium-nickel silicon thermocouples. The readings of the thermocouples were averaged automatically, and the results were used to adjust the temperatures in the furnace chamber in line with the ISO 834 standard fire curve (ISO 834-1, 1999) [20]. The ambient temperature was approximately 20 ^{o}C at the beginning of the fire resistance tests.

Before testing, parts of the joint were transmitted to the furnace using a crane with both ends of the square CFST columns fitted into the directional hinge supports, and then two extruded steel beam segments with one pair of stiffeners were mounted on the existing beam ends using high-strength bolts. For specimens under localized fire, only the bottom of the RC slab was exposed to fire, and the sides and top of the RC slab and the upper part of the square CFST column were wrapped by a layer of ceramic fibre blanket having a length of 3600 mm, breadth of 600 mm, and thickness of 50 mm, as shown in Fig. 5. For the specimens under global fire, all parts of the joint in the furnace chamber were exposed to fire. Other parts had ceramic fibre blankets positioned to protect the directional hinge supports and the connections between the beam segments and to keep the heat within the furnace chamber, as demonstrated in Figs. 4 and 5. To measure the development of axial displacement of the square CFST columns and vertical deflection of the beam ends, four and two displacement transducers, respectively, were placed at the corners of the upper directional hinge support and at the flange below the outside stiffeners, as shown in Fig. 4.

During fire heating, a constant axial compressive load and another vertical load were respectively applied to the top of the square CFST column and a location 100 mm away from the beam ends using a hydraulic jack. The joint specimens were exposed to heating following the standard fire curve (ISO 834-1, 1999) [20]. The tests ceased when the joint specimens reached their fire resistance limit. The joint specimens are considered to have achieved fire resistance when the deformation or deformation rate of the square CFST column or steel beam meet the ultimate conditions specified in the ISO 834 standard (ISO 834-1, 1999) [20], i.e. (1) for the square CFST column, when the axial displacement reaches $0.01H_{\mathrm{f}}$, or the axial displacement rate is greater than $3H_{\mathrm{f}}/1000$ per minute, where H_{f} is the fire exposure height of the column, which is 2750 mm in this study, and (2) for the steel beam, when the vertical deflection at the loading locations reaches $L_{\mathrm{b}}^2/(400h_{\mathrm{b}})$, or the vertical deflection rate is greater than $L_{\mathrm{b}}^2/(9000h_{\mathrm{b}})$ per minute. The minimum fire resistance of the square CFST column and steel beam is defined as the fire resistance of the joint specimens.

3. Test results and discussion

3.1. Overall observations and failure patterns

The observations from the fire tests indicated that the evaporation of steam from the RC slabs and the ends of the square CFST columns occurred approximately 10 min after fire initiation, and the surface colour of the steel beam and square CFST column gradually changed to dark red until fire resistance was reached. In general, there were two kinds of condition for the fire resistance of joints:

one, labelled beam failure (BF), occurred when the steel beam reached its fire resistance earlier than the square CFST column, and the other, labelled column failure (CF), occurred when the square CFST column reached its fire resistance earlier than the steel beam. The conditions for the fire resistance of the joint specimens is presented in Table 1. It can be seen that the dominant failure mode of the specimens with a beam load ratio (m) of 0.35 is BF, except for specimen JG1, and those with an m of 0.1 are dominated by CF, indicating that the fire resistance of the joints changes from BF to CF with a decrease in m. This is due to the fact that the beam of the joints with an m of 0.35 withstands greater load than that of the joints with an m of 0.1, therefore, earlier failure of the beam than the column is inevitable. For the specimen JG1, the material and loading defects produced in the column and beam may have dominated the failure process of the joint.

Figure 6 shows the failure patterns of the joint specimens after reaching fire resistance. It can be seen that, generally, tube buckling happens in the lower part of the square CFST columns for the cases under localized fire conditions; however, tube buckling appears in both the upper and lower parts of the square CFST columns for the cases under global fire conditions. Moreover, regardless of the fire scenarios and the ratio of linear stiffness of beam to column (k), there is lower flange buckling of the steel beam for the specimens with an m of 0.35, and no flange buckling of the steel beam occurs to the specimens with an m of 0.1. Additionally, there is a separation between the top flange of the steel beams and the RC slabs at the end of the slab for the joints with an m of 0.35 due to the increased loading effect. The comparison between the specimens for different fire scenarios and values of k indicates that, generally, k has a moderate effect on the failure patterns of the joints under localized fire conditions; however, for joints under global fire conditions, the tube buckling mode of the square CFST columns changes with the variation in k. This may be caused by the difference in the duration of heating while achieving fire resistance. It can also be found from Fig. 6 that despite the fire scenarios, in all tests buckles occur at a certain distance away from the connection zone in the tube of the square CFST columns, which is different from the observations in the fire tests of the single column, in which the tube buckles along its full height (Yang et al. 2017) [23]. This is primarily due to the constraint effect of the external ring plates and the relatively low temperatures within the connection zone (see section 3.2).

3.2. Temperature distributions

The comparison between the recorded chamber temperature (T) versus heating time (t) relationship for all tests and the ISO 834 standard fire curve (ISO 834-1, 1999) [20] is demonstrated in Fig. 7. It can be seen that within the initial 10 min, there is a certain degree of divergence between the $T-t$ curves of the furnace chamber and the standard fire curve. This is due to the fact that the gas supply of the furnace during the initial heating period cannot completely satisfy the required heating rate stipulated by the ISO 834 standard. After that, the $T-t$ curves measured in the furnace chamber are in very close proximity to the standard fire curve, except for specimens JL1, JG1, and JG4. In the tests for specimens JL1 and JG4, the furnace chamber loses some of its heat through the rectangular holes in the furnace wall because of the relatively large vertical deflection of the beam, and the gas supply in the furnace chamber of specimen JG1 is unstable.

Figure 8 plots the measured $T-t$ curves, which represent the thermal response of the joints, of eight joint specimens. The results for several positions are not given due to the malfunction of the corresponding thermocouples. It can be seen that the temperatures in the square CFST column and steel beam decrease with the decrease in the distance from the centre of the column and the lower surface of the RC slab, respectively. For the same heating time (t), the temperatures recorded at the positions in the square CFST columns (sections CJ and CN) are lower than those in the steel beams and RC slabs (sections BJ and BN) due to the larger endothermic effect of the core concrete in the square CFST columns than the RC slab on the steel beams. The recorded $T-t$ curves of the column sections in the composite joints are similar to the ones in the fire tests of the single column (Yang et al. 2017) [23]; however, due to the endothermic effect on the steel beam, concrete slab, and ring plate, a relatively low temperature was produced in the square CFST column as a component of the joints.

Figure 9 shows a typical comparison of the $T-t$ relationship of a structural member at different sections and positions. It is shown that the temperature within the connection zone (sections CJ and BJ) is lower than that outside the connection zone (sections CN and BN). This can be explained by the fact that the region within the connection zone is smaller than that outside the connection zone, which has a lower fire exposure area, and the temperatures of the beams (slabs) within the connection zone are also affected by the column, and vice versa. The results of the temperature distributions in this study are similar to the findings in the tests of Han et al. (2008) [16] and Song et al. (2010) [18].

3.3. Displacement versus time curves

The effect of the parameters on the axial displacement of the column (Δ_c), and the vertical deflection of the beam ends (δ_b) versus heating time (t) curves are shown in Fig. 10. It can be seen from Fig. 10(a) that the Δ_c-t curve of the square CFST column in the joint specimens is similar to that in the single column tests (Yang et al. 2017) [23], i.e. Δ_c increases and decreases with the increase in t mainly because of the expansion and strength degradation, respectively, of materials at high temperatures before and after reaching peak axial displacement; however, the maximum value of upward axial displacement of the square CFST column in the joints is lower than that in the fire tests of the single column (Yang et al. 2017) [23] due to the action of the loads on the beam, and the weight of the concrete slab. Generally, under the same k and m values, joints under global fire conditions demonstrate a larger peak upward Δ_c and a lower corresponding t than those under localized fire due to the increased fire exposure area, and regardless of the fire scenarios, k and m exert a moderate effect on the Δ_c-t curve before reaching the peak upward Δ_c, except for the specimens JL1 and JG1. This may be caused by the difference between the heating curve and the ISO 834 standard fire curve (see Fig. 7). However, the Δ_c-t curve of the joints with different parameters varies in different ways after achieving the peak upward Δ_c due to the difference in the tube buckling mode of the square CFST columns (see Fig. 6).

It is shown in Fig. 10(b) that, in general, the left- and right-side beams display different δ_b-t relationships and for the same heating time. The right beam end has a larger downward deflection than the left beam end, irrespective of the fire scenarios. This is because the more significant tube buckling of the square CFST columns is generally located at one side, resulting in a bow shape protruding toward the opposite side, as shown in Fig. 6. At the same time, the δ_b-t curves recorded at the left beam end also fluctuate with the variation in heating time, mainly because the beam is connected to the square CFST column. In general, k values and fire scenarios have a moderate effect on the δ_b-t curves; however, δ_b increases with the increase in m due to the increased loading effect.

3.4. Fire resistance

The experimental fire resistance ($t_{R,e}$) of the joints is presented in Table 1, and the effect of parameters on $t_{R,e}$ is shown in Fig. 11. It can be observed from Fig. 11 and Table 1 that, for the specimens under localized fire conditions, a larger m leads to a smaller $t_{R,e}$ due to the increased damage rate of the beams, and k has a moderate influence on $t_{R,e}$. When m is increased from 0.1 to 0.35 while the same value of k is maintained, 42.4%–44.1% lower $t_{R,e}$ is produced. However, for the specimens under global fire conditions, both m and k have a significant effect on $t_{R,e}$, and $t_{R,e}$ decreases with the decrease in m and k. This may be due to the fact that the variation in the fire exposure area of the joints with larger m and k values leads to a better load bearing mechanism. Specimens with an m of 0.1 have a $t_{R,e}$ approximately 35% lower than those with an m of 0.35, and similarly, specimens with a k of 0.2 possess a $t_{R,e}$ approximately 46% lower than those with a k of 0.3. Moreover, when $m = 0.1$, the joints under localized fire conditions demonstrate a larger $t_{R,e}$ than those under global fire; however, the fire scenarios have a moderate effect on the $t_{R,e}$ of the joints with an m of 0.35 except for specimen JG1, due to the obvious reduction in its heating curve from other specimens. Furthermore, the fire resistance of composite joints with the same column axial compression ratio is not consistent due to the change in m, k, and the fire scenarios, i.e. the variation in the parameters of the joints affects the fire performance and fire resistance of the square CFST column in the joints. Particularly, the square CFST column in the joints subjected to fire cannot reach its fire resistance limit when the beam in the joints reaches its limit state first.

4. Numerical simulation

4.1. General description

A three-dimensional FEA model was built using the ABAQUS software package (Simulia 2014) [24] to carry out the numerical simulation for the performance of the joints under combined loading and fire conditions. There were two analysis steps involved in the numerical simulation process—thermal analysis and mechanical analysis—and both steps used the same element meshing and node numbering.

4.2. Details of the FEA model

In the thermal analysis step, the thermal properties of steel beam, steel tube and steel rebars adopted were those provided in Lie (1994) [25], which had been successfully used in other temperature distribution analyses of the square CFST columns and beam to CFST column joints (Tan et al. 2012; Song et al. 2010; Song and Han 2014; Yang et al. 2017) [17-19,23]. The thermal properties of concrete given by Lie (1994) [25] were also adopted for the core concrete in the square CFST columns and the concrete of the RC slabs, and the free water in the concrete, measured by a moisture content of 5%, was also taken into account.

The uncoupled heat transfer analysis provided in the ABAQUS software package (Simulia 2014) [24] was used for the thermal analysis, and both convection and radiation were considered in the analysis. The convective coefficient, surface radiation emissivity and Stefan-Boltzmannn constant were adopted from the report by the European Convention for Construction Steelwork (ECCS) Technical Committee 3 (ECCS 1988) [26], and they were 25 W/(m^2·K), 0.5, and 5.67×10^{-8} W/(m^2·K^4), respectively. Furthermore, the thermal contact resistance between the steel tube and its core concrete in the square CFST columns was set to 0.01 $m^2 \cdot K/W$, based on the suggestion in Ghojel (2004) [27]. In the heat transfer model, the steel beams and steel tube of the square CFST columns were simulated by three-dimensional 4-node quadrilateral shell elements (DS4), and the endplates, core concrete in the square CFST columns, and concrete of the RC slabs were modelled by three-dimensional 8-node linear brick elements (DC3D8). All steel rebars were simulated by the 2-nodes truss elements (DC1D2). The meshing and boundary conditions of the joint model are shown in Fig. 12. The fire exposure height of the square CFST columns was the same as that in the tests, and the recorded fire heating curves from the tests were adopted as the input data.

In the mechanical analysis step, the steel plasticity model in the ABAQUS software package (Simulia 2014) [24] was used to capture the nonlinear behaviour of the steel beams, steel tube of the square CFST columns, and steel rebars under high temperatures, and the model used von Mises yield surfaces with the associated plastic flow. The tabulated data for true stress versus logarithmic plastic true strain converted from the engineering stress versus engineering strain relationship in Lie (1994) [25] were used for the steel components, and the high-temperature creep of steel was taken into account using the model suggested by Fields and Fields (1991) [28].

The damaged plasticity model in the ABAQUS software package (Simulia 2014) [24] was adopted to simulate the complicated nonlinear behaviour of core concrete in the square CFST columns and concrete of the RC slabs under high temperatures, and the required tabulated data of stress versus inelastic strain of concrete were obtained based on the engineering stress versus engineering strain relationship suggested in Han and Song (2016) [29] and Lie (1994) [25], respectively. The model for both thermal creep and transient thermal strain of concrete dependent on current temperature and stress proposed by Anderberg and Thelandersson (1976) [30] was considered in this study. For concrete under tension, the stress versus fracture energy model in the ABAQUS software package (Simulia 2014) [24] was adopted to describe the tension stiffening effect. The cracking stress was equal to 10% of peak compressive stress, and the fracture energy was determined using the equation in Han and Song (2012) [29].

Furthermore, the elastic modulus of steel and concrete under high temperatures was the initial slope of the engineering stress versus engineering strain curves. The Poisson's ratio of steel was set to be 0.3 at both ambient and high temperatures. The Poisson's ratio of concrete was set to 0.2 at ambient temperature and was reduced to 50% of its ambient temperature value at a temperature of 150 ^{o}C, and then to 0 at a temperature of 1200 ^{o}C (Izzuddin et al. 2002) [31]. The thermal expansion coefficient of steel and concrete in Lie (1994) [25] was incorporated in the FEA model.

After creating the parts for all joint components, the following procedure was used to build the joints: (1) in the Part module of ABAQUS, a partition between the square steel tube and its core concrete was made at the intersection of beam (including slab) and column, and the steel beams and ring plates were merged together using a function available in ABAQUS; and (2) in the Assembly module of ABAQUS, firstly, the bottom centre of the square steel tube was translated to the origin of the local coordinates, and the core concrete was then assembled to its outer tube; secondly, the steel beam with ring plates and concrete slab were sequentially assembled to the square steel tube using the translation function; thirdly, the longitudinal and distribution bars were assembled to the concrete slab using the "array operation" based on their designed positions; and finally, the rigid plates (replacing the directional hinge supports in the tests for the purpose of simplifying the calculation) and loading plates were assembled to the square CFST column and steel beam, respectively.

The steel beams and steel tube were simulated using the three-dimensional 4-node reduced-integration shell elements (S4R), and the endplates, core concrete in the square CFST columns, and concrete in the RC slabs were modelled by the three-dimensional 8-node reduced-integration brick elements (C3D8R). All steel rebars were simulated by the 2-node truss elements (T3D2).

For the square CFST columns, the surface interactions along the normal and tangential directions between the steel tube (end plate) and its core concrete were modelled using 'Contact interaction' from ABAQUS, and they were set to 'Hard contact' and 'Coulomb friction', respectively, with a friction coefficient of 0.6 (Song et al. 2018; Han and Song 2012) [5,29]. The interaction between the endplates and steel tube was defined as the 'Coupling' between the shell and solid elements. The interactions between the steel bars and concrete of the RC slab, and the steel beam and RC slab were realized through 'Embedded' and 'Tie' constraints, respectively. Moreover, to consider the effect of overall geometric imperfections on the column, an initial eccentricity (e_i) of $L_0/1000$ to the centroidal axis was applied at both endplates of the square CFST columns, as shown in Fig. 12, with L_0 equal to the distance between the rollers in the top and bottom directional hinge supports.

The boundary conditions for the joints are shown in Fig. 12, where N_F and P_F are the constant load applied on the top of the column and at the beam ends to replicate the tests. To reproduce the boundary conditions in the tests, four reference points (RPs) coupled with two column endplates and two beam ends were defined. The two RPs on the column were located at the symmetry axis, and the two RPs on the beam were located at half the height of the web plate. The translation of the RP on the bottom endplate of the column was constrained to three degrees of freedom (DOFs). For the RP on the top endplate of the column, constraints in the translational X and Y direction were applied. Both RPs at the beam end had the same boundary conditions, i.e. the translational displacements in the Y direction, and rotations in the X and Z directions were constrained. Furthermore, geometric nonlinearity effects were considered, as large lateral deflections were observed in the tests.

In the analysis, the constant axial compressive load (N_F) and vertical load (P_F) were first applied at the top of the column and to the beam ends, respectively, and then thermal distribution was employed by reading nodal temperatures obtained from the thermal analysis. The thermal responses of each joint were obtained by the Simpson-Newton's method.

4.3. Verification of the numerical model

Fig. 13 demonstrates the predicted failure patterns of the joints during fire resistance testing, where the von Mises stresses of steel tube in the square CFST columns and steel beams, and the maximum principal plastic strains of the RC slabs are presented. It can be seen from the comparison between

Fig. 13 and Fig. 6 that, for joint specimens JL2, JL4, JG2, and JG4, both the predicted and observed failure patterns, which are the tube buckling in the square CFST columns with relatively low von Mises stresses in the steel beams and the predicted positions of the large buckling of steel tube of the square CFST columns, generally agree well with the observed results. For the joint specimens JL1, JL3, and JG3, the predicted and observed failure patterns are the flange buckling of beams with relatively low degree of local buckling and small von Mises stresses in the steel tube of the square CFST columns, and the predicted deformation mode of the steel beams is generally in good agreement with the measured deformation mode. The predicted positions of the maximum von Mises stress in the steel beams was located at the end of the RC slab due to stiffness mutation (from composite beam to steel beam), which is slightly different from those observed in the tests. This can be explained by the fact that the separation between RC slabs and steel beams in the tests causes the stiffness mutation position to move inward.

As is displayed in Fig. 13, under the same conditions, joints with a larger m demonstrate a larger maximum principal plastic strain in the RC slab owing to the increased loading effect, and the joints under localized fire conditions have a larger maximum principal plastic strain in the RC slab than those under global fire conditions due to the larger temperature gradient. For joints exhibiting BF, a larger k results in a smaller maximum principal plastic strain in the RC slab; however, for the joints with CF, a larger k leads to a larger maximum principal plastic strain in the RC slab. Moreover, the region of the RC slab having greater distance from the connection zone is easier to crack. The predicted conditions for fire resistance of the joint specimens is listed in Table 1. It can be seen that only specimen JG1 has a prediction result different from that recorded in the tests. This is caused by the fact that the real heating curve of specimen JG1 diverges greatly from the ISO 834 standard fire curve, and the FEA model in this study cannot account for the effect of the temperature fluctuation on the properties of the materials.

The typical comparison between the predicted and observed member $T-t$ curves of the joint specimens is given in Fig. 14. It can be observed that the trends of the predicted $T-t$ curves generally agree well with the observed ones, although there is a certain difference between them. This may be caused by the discrepancy between the design location of the thermocouples and their actual position in the tests. Figure 15 shows a plot of the predicted temperatures (T_c) against the measured results (T_e) by the functional thermocouples, where AV and SD stand for the mean and standard deviation of T_c / T_e, respectively. The predicted temperatures generally exhibited good agreement with the measured ones, with an AV and SD of 0.957 and 0.214, respectively. However, a relatively large discrepancy between the numerical simulation and test results is also demonstrated in Fig. 15. This is due to the fact that (1) the water evaporation inside the concrete is irregular; 2) the convection and emissivity coefficients of the joint surface in the connection zone might be lower than those in the non-connection zone due to the existence of the adjacent beam and column; 3) the thermal response of the cracked concrete is discontinuous; and 4) the position of the thermocouples in the specimens may deviate from their design positions. These facts were not well incorporated into the FEA model. However, it can be seen that, in general, the difference in temperature prediction has moderate impact on the subsequent mechanical analysis of the joint model, as the average error between the predicted and measured temperatures is within 5%.

The comparison between the predicted and measured axial displacement of the square CFST column (Δ_c) versus heating time (t) curves is illustrated in Fig. 16. It can be seen that a reasonably good agreement is obtained between the predicted and measured $\Delta_c - t$ curves, and both results have change stages similar to those in the single column tests (Yang et al. 2017) [23]. However, certain differences are also observed, especially during the stage after achieving peak upward displacement. This is possibly due to unexpected environmental factors, material defects, and variation in the temperature measuring points in the tests, which cannot be completely reproduced in the FEA model.

Figure 17 shows a comparison between the predicted and recorded vertical deflection of the beam ends (δ_b) versus heating time (t) curves. It can be seen that the FEA model can successfully capture the characteristics of the vertical deflection of both left and right beam ends. Generally, the predicted $\delta_b - t$ curves agree well with the recorded results in the initial and middle development phase; however, in the latter development phase, the predicted $\delta_b - t$ curves show a certain deviation from the measured ones due again to the fact that some experimental factors cannot be considered in the FEA model.

The predicted fire resistance ($t_{R,p}$) of eight joint specimens is presented in Table 1, and the comparison between $t_{R,p}$ and $t_{R,e}$ is plotted in Fig. 18. It can be seen from Table 1 and Fig. 18 that, in general, a good agreement between the predicted and experimental results is attained, although a certain discrepancy between $t_{R,p}$ and $t_{R,e}$ is observed for a few of the joints.

From the above verifications, it can be concluded that the FEA model in this study may reasonably predict the fire performance of steel beam to square CFST column composite joints using RC slab.

4.4. Effect of typical parameters on the fire resistance

To prove the versatility of the FEA model, and to inspect the influence of parameters which are not considered in the tests, the validated FEA model was further adopted to investigate the effect of typical parameters on the fire resistance (t_R) of steel-concrete composite beam to square CFST column joints subjected to ISO 834 standard fire. To incorporate the effect of the adjacent components in a non-fire zone, a 2D frame joint model with axial and flexural constraints at the beam ends was constructed, as shown in Fig. 19. In the FEA model, the axial and flexural springs were set to reproduce the axial (K_A) and flexural (K_F) constraint stiffness at the beam ends, which were calculated using the suggestions given by Huang and Tan (2006) [32]:

$$K_A = \chi \cdot \frac{(EI)_b}{L_b} = \left(\frac{1}{k} \cdot \frac{24j}{H_c^2} \right) \cdot \frac{(EI)_b}{L_b} \tag{4}$$

$$K_F = \beta \cdot \frac{(EI)_b}{L_b} = \left(\frac{2}{k} + \frac{6\alpha}{4-\alpha^2} \right) \cdot \frac{(EI)_b}{L_b} \tag{5}$$

where χ and β are respectively defined as the axial and flexural constraint stiffness ratio, j is the total number of bays on the right-hand side of the fire exposure compartment, and α is a factor related to the constraint condition at the beam ends.

The basic input conditions were: $B_c \times t_c = 500$ mm×12 mm ($\alpha_c = 0.1$), $h_b \times b_f \times t_w \times t_f = 600$ mm × 300 mm × 20 mm × 20 mm, $b_{slab} \times t_{slab} = 1640$ mm × 120 mm, $H_c = 3900$ mm, $L_b = L_{slab} = 6000$ mm, $f_{c,c} = 50$ MPa, $f_{c,b} = 32$ MPa, $f_{y,c} = f_{y,b} = 345$ MPa, $f_{y,r} = 335$ MPa, $a_c = a_b = 10$ mm, $n = 0.6$, $m = 0.4$, $k = 0.47$, $\chi = 20.05$ m^{-2}, and $\beta = 7$, where α_c is the steel ratio of the square CFST (Han et al. 2014) [1], $f_{c,c}$ and $f_{c,b}$ are the cylindrical compressive strength of core concrete in the square CFST column and the concrete in RC slab, respectively, $f_{y,c}$, $f_{y,b}$, and $f_{y,r}$ are the yield strength of the steel tube in the square CFST column, steel beam, and steel bars in the RC slab, respectively, and a_c and a_b are the fire protection thickness of the square CFST column and the steel beam, respectively. Moreover, the dimensions of RC slab and the parameters of the steel bars were kept constant to simplify the modelling.

Figure 20 displays the calculated fire resistance (t_R) of the composite joints subjected to localized fire with different parameters. It can be seen that, generally, t_R increases with an increase in B_c, α_c, a_c, $f_{c,b}$, a_b, and k, and a decrease in H_c, $f_{c,c}$ and m. $f_{y,c}$, $f_{y,b}$, χ, and β have a moderate effect on t_R. Furthermore, when n is less than 0.4, t_R slowly increases with an increase of n; however, when n is greater than 0.4, t_R quickly decreases with an increase of n. The findings of the influence of the key parameters on fire resistance of this type of joint provide a basis for follow-up testing and theoretical study.

5. Conclusions

In this study, experimental and numerical investigations are conducted for H-shaped steel beam to square concrete-filled steel tube (CFST) column composite joints with RC slab subjected to localized and global fire conditions. The following conclusions can be drawn:

(1) According to the test results, the failure mode of the composite joint under fire conditions changes from BF to CF with the decreasing load ratio of beams (m). The main failure patterns of the composite joint include tube buckling of the square CFST columns, flange buckling of the steel beams, and separation between the top flange of the steel beams and the RC slabs, and these patterns are determined by the various parameters in the tests.

(2) For all joint specimens, the temperatures in the square CFST column and steel beam increase with the increase in the distance from the centre of column and from the lower surface of the RC slab, respectively, and the temperatures of the structural members within the connection zone are lower than those in the other regions.

3) Generally, the axial displacement (Δ_c) versus heating time (t) curve of the square CFST column in the joint specimens is similar to that in the single column tests, and under the same beam-to-column linear stiffness ration (k) and m values, the joints under global fire conditions demonstrate a larger peak Δ_c and a lower corresponding t than those under localized fire conditions. Meanwhile, the left and right beam end of the joint specimens display different vertical deflection (δ_b) versus t relationship. The fire scenarios and k values exert a moderate effect on the δ_b-t curves; however, δ_b increases with the increase in m due to the increased loading effect.

(4) For joint specimens under localized fire conditions, a larger m leads to a smaller fire resistance ($t_{R,e}$), and when m is increased from 0.1 to 0.35 with the same k value, a 42.4%–44.1% reduction in $t_{R,e}$ is observed. For joint specimens under global fire conditions, both m and k have a significant effect on $t_{R,e}$, and $t_{R,e}$ decreases with a decrease in m or k. When $m = 0.1$, the joint specimens under localized fire conditions demonstrate a greater $t_{R,e}$ than those under global fire conditions; however, the fire scenarios have a moderate effect on the $t_{R,e}$ of joint specimens with an m of 0.35.

(5) A nonlinear FEA model was built using the ABAQUS package, and the modelling responses

of the joints subjected to ISO 834 standard fire were in good agreement with the experimental results. Parametric analysis of the steel-concrete composite beam to square CFST column joints subjected to ISO 834 standard fire was then performed, and the key parameters that influence the fire resistance of this kind of joint were identified.

It should be noted that an important issue of joint behaviour within a composite frame structure in fire is that the forces on each joint component change with time, and this phenomenon was difficult to capture in both the experimental and numerical results presented, due to the limitations of both the experimental facilities and the numerical model. However, the overall failure modes, temperature distribution, and fire resistance of the joints to be used by design engineers were well captured. To solve this problem in the future, based on the research outcomes of this study, thermal-mechanical coupling research on the fire performance of CFST composite joints must be further carried out to reasonably consider the effect of the time-related joint forces.

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