Simulation of Mechanical Properties of Special-Shaped Concrete-Filled Steel Tubular Column and Steel Beam Joints after Fire

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Abstract. To study fire after the mechanical performance of steel girder node special-shaped concrete-filled steel tube column, based on standard ISO - 834 litres of cooling curve, the node temperature field model was established based on finite element software ABAQUS, the compute nodes in the overall uniform temperature field under fire as a result, the reasonable choice of fire after the steel and concrete constitutive model, the temperature field results into the node stress model, considering the factors that influence the whole effect of fire loading in low cycle, the nodes of the finite element model, and contrast analysis of the temperature after the fire of the node and hysteretic performance and ultimate bearing capacity. The results show that the failure modes of special-shaped CFST column-steel beam joints at room temperature and after fire are the same, and the ultimate bearing capacity of the joints after fire decreases significantly by 14.88% compared with that at room temperature.

Keywords. Concrete filled steel tube; Cross node; fire; The temperature field.

1. Introduction
Fire is one of the common disasters in people’s lives. Once a fire occurs, it will pose a huge threat to people’s lives and property. Building fires occur most frequently, The losses caused are also the most serious. In recent years, concrete-filled steel tube structure has been used more and more widely in building structures due to its excellent mechanical properties. Its fire risk is also increasing gradually. The research on the properties of concrete-filled steel tube after fire will provide a basis for post-disaster repair and reinforcement [1]. As a force transfer hub of the frame structure, Transfer the bending moment and shear force of the beam and column. In order to further study the mechanical properties of special-shaped concrete-filled steel tubular structures after fire, In this paper, the temperature field and mechanical properties of concrete-filled steel tube cross column and steel beam joint after fire are simulated and analyzed. It provides a theoretical basis for the reinforcement and repair of such joints after disaster.

2. Model Design Scheme
The sample diagram of node model is shown in figure 1. The node attaches the side plate flat to the node domain, It is above the I-beam and extends a certain distance along its axis. The side panels strengthen the node domain obviously, The construction of the node is convenient, the force
transmission is clear. This paper designs two models, Model size parameters are shown in table 1. JD1 is the node analysis model at room temperature. TJD1 is a thermodynamic coupling model after high temperature action.

| Specimen number | Section size of column | Frame beam dimensions | Axial compression ratio | th(min) | The working condition of the fire |
|-----------------|------------------------|-----------------------|------------------------|---------|----------------------------------|
| JD1             | Web limb:300×100×5;Flange limb:100×100×5 | 250×100×4×6          | —                      | 30      | The nodal model is evenly exposed to fire |
| TJD1            |                        |                       | 0.3                    |         | From the fire                     |

3. Fire Action Simulation of Node Model

3.1. Thermal Parameters of Node Model Materials

Under the action of fire, Heat is exchanged through heat conduction inside the cross - shaped concrete-filled steel tubular column-steel beam joints. In order to ensure the accuracy of heat transfer simulation, in this paper, the thermal conductivity of steel was calculated according to the formula (1) given by Lie [2] and the formula (2) for calculating the thermal conductivity of ordinary concrete suggested by the European Code [3].
According to the recommendation of European Code [3], the specific heat calculation formula (3) of steel and the specific heat calculation formula (4) of ordinary concrete are selected.

\[
K_s = \begin{cases} 
-0.022T + 48 \frac{W}{m^2\cdot{\textcircled{C}}} & \text{°C} \leq T \leq \text{900 °C} \\
28.2 \frac{W}{m^2\cdot{\textcircled{C}}} & T > \text{900 °C} 
\end{cases} 
\] (1)

\[
K_c = 0.012\left(\frac{T}{120}\right)^2 - 0.24\left(\frac{T}{120}\right) + 2 & \text{20 °C} \leq T_s \leq \text{1200 °C} 
\] (2)

Where, \( T \) is temperature (°C), and the specific heat unit is J (kg·°C).

The mass density of steel and concrete is less affected by temperature, take 2300kg/m³ and 7850 kg/m³, respectively.

### 3.2. Temperature-Time Curve of Composite Heating

In this paper, the rising and cooling curve suggested by the national standard ISO-834[4] is adopted to simulate the real situation of fire. The specific expression (5) is as follows:

Heating stage : \((T \leq t_h)\)

\[
T = T_0 + 345\log_{10}(8t + 1) 
\] (6)

Cooling period : \((T > t_h)\)

\[
T = \begin{cases} 
T_s - 10.417(t - t_h) & t_h \leq 30 \text{ min} \\
T_s - 4.167(3 - \frac{t_h}{60})(t - t_h) & 30 \text{ min} \leq t_h \leq 120 \text{ min} \\
T_s - 4.167(t - t_h) & t_h > 120 \text{ min} 
\end{cases} 
\] (5)

Where, the temperature unit is °C

\( t \): total cooling time

\( T \): temperature at time \( T \)

\( T_0 \): initial temperature

\( T_s \): critical temperature for rising and cooling

### 3.3. Analysis of Cross Section Temperature Field

In order to better analyze the temperature variation rule at different locations of nodes, a feature point of each component at the horizontal midline section of the node is selected as the research object. The feature points of side plate, steel tube column, core concrete and steel beam are shown in figure 2. Corresponding to feature points 1, 2, 3 and 4, and extracting the temperature of feature points under the action of fire in the whole process of rising and cooling. Draw the time-temperature curve as shown in figure 3.
As can be seen from figure 3, the time-temperature curve of each component of the node model is basically consistent with the law of the ISO-834 standard lift-temperature curve. The time-temperature curve of feature point 4 is the most consistent. The maximum temperature at feature points 1, 2, and 3 decreases in turn. It takes longer to reach the maximum temperature. In the heating process of feature point 3, the temperature of feature point 3 is always low. At the cooling stage of feature point 3, the temperature of feature point 3 is higher than that of other parts in the figure. This is due to the close contact between the side plate and the steel tube column and the core concrete. In the heating process, evaporation of water from the core concrete consumes heat and cools the core concrete. The thermal inertia of the concrete will cause the temperature rise and drop of the cross concrete filled steel tube to lag behind that of the steel beam.

Based on reasonable setting of material thermal parameters, fire conditions and cell mesh, the heat transfer is simulated by integrating heat transfer coefficient and thermal radiation coefficient under the temperature load of the fire surface. The finite element analysis of the whole fire process of concrete-filled steel tube cross column and I-beam joint is calculated. According to the finite element calculation results, finite element results of temperature field in TJD1 model at $t_h$ time, 30min cooling and 60min cooling are shown in figure 4, the unit of temperature is °C.
As can be seen from the figure above, the temperature at different positions of the node model varies greatly. Mainly because of the steel pipe and concrete two kinds of material thermal performance is different. At the th moment of the ISO-834 standard lift-temperature curve, the steel beam reaches the highest temperature of 818.2℃. At this point, on the section of the CFST column, the side plate has the highest temperature, about 750 ℃, The closer you get to the center of the column, the colder the core concrete gets, the temperature of the core concrete near the corner of the steel tube column is higher. This is because the concrete near the corner of the steel tube column has a large contact area with the steel tube column, it also transfers heat faster when exposed to fire, so the core concrete at the corner is warmer than the other places. Combined with Figure 3, it can be seen that, When the total time of exposure to fire is 60min, At this point, the temperature of the ISO-834 standard temperature curve has been lowered for 30min, By this time the steel columns, concrete and side slabs had begun to cool down, The core concrete is still heating up, When the node model enters the later cooling period (the total time t is greater than 120min), Core concrete is the hottest. It is the thermal inertia of concrete that leads to the difference of rising and cooling time of each component of the joint model.

3.4. Simulation of Mechanical Properties of Joints After Fire

3.4.1. Selection of Material Constitutive. Based on the research results in literature [1], This paper ignores the effect of the whole fire process, In other words, the effect of rising and cooling on the mechanical properties of the joints after fire is not considered. Cao Wenxian[5] was selected to study the constitutive relationship of steel in concretelined steel tube at high temperature. Formula (6) is as follows:

\[
\sigma = \begin{cases} 
E_s(T_{max}) \cdot \varepsilon & \varepsilon \leq \varepsilon_s(T_{max}) \\
 f_s(T_{max}) + E_s(T_{max}) \cdot [\varepsilon - \varepsilon_s(T_{max})] & \varepsilon > \varepsilon_s(T_{max}) 
\end{cases}
\]

Where, \(E_s\): elastic modulus of steel at room temperature

In Equation (6),
\(E_s(T_{max})\): the highest temperature in the history of the material,
\(E_s(T_{max}) = E_s; \quad E_s(T_{max}) = 0.01 E_s(T_{max});\)

\(\varepsilon(T_{max}) = f_s(T_{max}) / E_s(T_{max})\)
E (T_{max}): Elastic modulus of steel after high temperature, Es=2.06×10^3 MPa;
εy (T_{max}): yield strain of steel at high temperature
f_y (T_{max}): yield strength of steel after high temperature.

\[ f_y (T_{max}) = \begin{cases} f_y & T_{max} > 400^\circ C \\ f_y [1 + 223 \times 10^4 (T_{max} - 20) - 588 \times 10^2 (T_{max} - 20)^2] & T_{max} > 400^\circ C \end{cases} \]

f_y: yield strength of steel at room temperature

The mechanical properties of concrete change after high temperature, Lin Xiaokang [6] modified the peak stress and peak strain of the concrete compression constitutive model. After modification, formula (7) is as follows:

\[ \sigma_o = \frac{f'_{c}}{1 + 2.4(T_{max} - 20) \times 10^{-10}} \left( \frac{N}{mm^2} \right) \]

\[ \varepsilon_c = (1300 + 12.5 \cdot f'_{c}) \cdot 10^{-6} \cdot [1 + (1500(T_{max} + 5T_{max}^2)) \times 10^{-6}] \]

Guo Zhenhai and Li Wei [7] based on the study of concrete after high temperature, the peak tensile stress and peak strain of concrete are modified. Formula (8) is modified as follows:

\[ \sigma_p = 0.26 \cdot (1.25 \cdot f'_{c}) \cdot (1 - 0.001T_{max}) \]

3.4.2. Establishment of Node Simulation Model. The joint model consists of a cross-shaped steel tube column, core concrete, side slabs and steel beams. It can save the calculation time and ensure the accuracy of calculation. All components are linear reduction integral element (C3D8R). The steel pipe and concrete use face to face contact, The tangential behavior adopts the Coulomb friction model provided by Abaqus, The friction coefficient at room temperature is 0.6, The friction coefficient after high temperature is 0.25 [8], The normal behavior selects the ABAQUS default hard contact.

In this paper, the column end loading method recommended by JGJ101-2015 Code for Seismic Test Methods of Buildings [9] was selected. The loading diagram is shown in Figure 5. The top surface of the column is coupled to the reference point RP1. Constrain its translation in the y direction and its rotation in the y and z directions, By applying pressure on the top surface of the column to simulate the axial force at the top of the column, A reciprocating horizontal displacement was applied to the reference point RP1 to simulate pseudo-static loading. The displacement increment is 5mm. The bottom of the column is coupled to the reference point RP2, It can only rotate about the Y-axis, The remaining degrees of freedom are all constrained. The left and right beam end faces are coupled to RP3 and RP4, respectively. Constrain y and z translation and x and z rotation.

Figure 5. Schematic diagram of model loading.

3.4.3. Initial Temperature Conditions. Since the uncoupled thermal analysis is carried out in this paper,
That is, without considering the effect of high temperature on the initial stress and strain inside the component, the temperature at a time in the temperature field ODB file was imported into the stress analysis model. Then the program determines the temperature of each node, and give the corresponding material properties.

According to the material constitutive relation model after high temperature, the stress-strain model after high temperature is only related to the highest historical temperature of the material. Because of the thermal inertia of concrete, each element of the node model cannot reach the maximum temperature at the same time. As the steel heats up, the temperature of concrete increases with the increase of the temperature of steel on the fire surface. The steel begins to enter the early cooling period, the concrete will still be warming up for a certain amount of time. When the steel has cooled to room temperature, steel is still in the cooling section, and keep it at a certain temperature. The nodes selected in this paper are designed according to the principle of "strong nodes and weak components". The joint failure position is on the steel beam. Therefore, the material properties of steel have a greater impact on the stress analysis model. In this paper, the joint temperature when the steel beam temperature reaches the highest temperature is taken as the initial temperature condition. That is, \( t \) is the cross section temperature at the moment when 30min.

4. Comparison of Node Simulation Results at Normal Temperature and After Fire

4.1. Model Failure Process

Mises stress nephograph of JD1 model loading process is shown in Figure 6. As can be seen from figure 6 (a), when the horizontal displacement is loaded to 25.32mm, beam-column connection parts, side plates and steel beam flanges begin to yield, at this point, the component enters the yield phase, the rest parts of the model were all in the elastic stage. When the horizontal displacement is pushed to 73.97mm, the ultimate load is 122.48kN. As the load continues, the flange deformation at the variable section position of the side plate overhanging steel beam is serious. When the horizontal displacement is loaded to 108.76mm, the load drops to 104.11kN, the capacity dropped to 85% of the ultimate capacity, loading finished. At this point, the failure state is shown in figure 6 (b).

![Stress nephogram of JD1 Mises](image)

(a) The yield state  (b) Damage state

Figure 6. Stress nephogram of JD1 Mises.

Mises stress nephograph during loading process of TJD1 model is shown in figure 7. As can be seen from figure 7 (a), when the horizontal displacement is loaded to 22.89mm (the same direction as the X coordinate in ABAQUS is positive, and the opposite direction is negative), the slope of skeleton curve changes obviously, The connecting parts of the upper and lower flanges of the steel beam and the steel tube column began to yield. TJD1 enters the yield phase, the steel tube column, core concrete and other parts of the steel beam were observed to be in the elastic stage. When this continues to load to 54.53mm, the maximum load is 104.25kN, the limit state is shown in figure 7 (b).
By observing the JD1 and TJD1 corruptions, Both JD1 and TJD1 are "beam hinge failure", That is, the flange of the steel beam at the outstretched end of the side plate yields first. As the displacement is further loaded, Deformation and bearing capacity gradually increase, Until a cross section plastic hinge is formed at this position, The node is broken. This suggests that the failure mode of the node model did not change after high temperature.

4.2. Skeleton Curve

The skeleton curve of the model is shown in figure 8. As can be seen from the figure, Compared with the finite element results of the nodal model at room temperature, The initial stiffness is basically unchanged, This is because the elastic modulus of steel and concrete is greatly reduced at high temperatures. After cooling to room temperature, The elastic modulus recovered obviously. The ultimate bearing capacity of T1JD1 decreased significantly compared with JD1. That's a 14.88 percent decrease. This means that when exposed to fire, This means that when exposed to fire, This is due to the effect of the whole fire, The mechanical properties of steel and concrete have been reduced to a certain extent, The extent of the drop depends on the record high temperature experienced by the material.

5. Conclusion

(1) After the whole process of rising and cooling is experienced by the side plate connecting the cross-shaped CFST to steel beam joints, The section temperature varies greatly in different positions. The time-temperature curve of steel beam characteristic points is in good agreement with the ISO-834 standard fire rise and drop curve. Because of the thermal inertia of concrete, The rising and cooling stages of each node component are not consistent. When the steel beam has cooled, The inside of the steel tube section may still be in the heating stage.

(2) Under normal temperature and high temperature, the failure mode of the side plate connecting the concrete-steel tubular cross beam joints is the same. That is, the flange of the steel beam at the
outstretched end of the side plate yields first. As the displacement is further loaded, Deformation and bearing capacity gradually increase, Until a cross section plastic hinge is formed at this position, The node is broken. This indicates that the inhomogeneity of temperature field does not lead to the change of failure mode of such nodes. However, its ultimate bearing capacity decreased significantly, The ultimate bearing capacity of T1JD1 is 14.88% lower than that of JD1.

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