Study on the axial compression performance of partially prefabricated steel concrete columns after fire

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Abstract: In combination with the advantages of steel reinforced concrete columns, high strength and high performance concrete and precast assembled structures, a new type of partially precast assembled steel reinforced concrete columns (SRHCC) was proposed in this paper. In order to study its post-fire axial compression performance, the finite element analysis software ABAQUS was used to simulate its temperature field under standard fire based on experiments. The results show that the core temperature of the specimen increases with the increase of the thickness of the section steel protective layer and decreases with the increase of the firing time. On this basis, the limit equilibrium theory was used to propose the formula for calculating the residual carrying capacity of SRHCC after fire. The calculated results were close to the test results and tended to be safe.

1. Introduction

Based on the advantages of reinforced concrete and precast concrete, this paper proposed a new partially precast reinforced concrete composite column (SRHCC). The section layout is shown in Fig.1(a). SRHCC is composed of precast part of high-strength concrete and cast-in-place part of ordinary concrete. The precast part is composed of cross steel, longitudinal reinforcement, continuous stirrup and high-strength concrete. The cast-in-place section consists of ordinary concrete. Because the fire will cause different degrees of damage to the structural members, at present, scholars at home and abroad have carried out more research on the performance of steel reinforced concrete columns under high temperature fire. In order to study the axial compression performance of SRHCC after fire, based on existing test results [1], finite element analysis software ABAQUS was used to simulate its temperature field under standard fire. The influence of the strength of cast-in-place concrete, hoop spacing and section size on the temperature field distribution of specimens is studied in order to provide reference for engineering design. The design parameters of specimens in reference [1] are shown in Table 1.

![Image](chart.png)
2. Cross section temperature field simulation

2.1. The basic assumptions

In order to facilitate the fire simulation calculation, the following assumptions are made based on the test results: 1) No bond slip between the section steel and concrete in each specimen; 2) There is no heat source inside the specimen; 3) Steel bar, section steel and concrete are isotropic materials; 4) Excluding the influence of sealing plate inside the specimen; 5) Excluding the thermal resistance of contact between the materials.

2.2. Thermal properties of materials

In this paper, the material thermal performance parameters of steel adopted the values recommended in EC3 [2] and EC4 [3] of the European specification, the material thermal performance parameters of ordinary concrete adopted the values recommended in EC4 of the European specification, and the thermal conductivity and specific heat capacity of RPC adopted the values recommended by Zheng Wenzhong. [4].

2.3. Modeling and calculation result analysis

Specimen sections and concrete unit adopts DC3D8 thermal analysis unit, longitudinal reinforcement and stirrup used DC1D2 thermal analysis unit, by the fire before the specimen in the indoor environment, set the initial temperature is 20°C, specimen meshing model shown in figure 1(b), cross section in the temperature contours as shown in figure 2, measuring point measured temperature curve compared with simulated temperature curve as shown in figure 3.

As can be seen from FIG. 2, the temperature field of the specimen presents a symmetric curved edge square temperature gradient distribution, and the closer to the center of the specimen, the closer to the circle. The temperature at the corner is higher than that in the middle of the specimen at the same level. This is because the concrete at the corner is affected by the temperature in two directions and is the weak part of the specimen's mechanical performance after the fire. At the end of the test (120min), the

| Specimen no. | Specimen size (mm) | Prefabricated thickness (mm) | Core concrete | Steel content | Hot rolled section specifications | Stirrup spacing (mm) |
|--------------|--------------------|-----------------------------|---------------|--------------|----------------------------------|----------------------|
| SRHCC30-2    | 300×300×825        | 62.5                        | C20           | 4.9%         | HN175×90×5×8                     | 40                   |
| SRHCC30-5    | 300×300×825        | 62.5                        | C50           | 4.9%         | HN175×90×5×8                     | 40                   |
| SRHCC30-10   | 300×300×825        | 62.5                        | RPC100        | 4.9%         | HN175×90×5×8                     | 40                   |
| SRHCC30-0-H  | 300×300×825        | 62.5                        | —             | 4.9%         | HN175×90×5×8                     | 120                  |
| SRHCC30-2-H  | 300×300×825        | 62.5                        | C20           | 4.9%         | HN175×90×5×8                     | 120                  |
| SRHCC25-2    | 250×250×690        | 62.5                        | C20           | 5.1%         | HN125×60×6×8                     | 40                   |
| SRHCC25-0    | 250×250×690        | 62.5                        | —             | 5.1%         | HN125×60×6×8                     | 40                   |
| SRHCC35-0    | 350×350×960        | 75.0                        | —             | 4.2%         | HN200×100×5.5×8                  | 40                   |
temperature level of core concrete was still low. The larger the section steel protective layer is, the lower the temperature inside the specimen is, and the thickness of the concrete protective layer has an obvious effect on the core concrete and section steel.

As can be seen from FIG. 3, the numerical simulation curve agrees well at 0-100°C, and the simulated data after 100°C is higher than the measured temperature. This is because at about 100°C, moisture absorption and evaporation in concrete are not taken into account in the numerical simulation. The simulated temperature of the measuring points d-42 and d-60 close to the surface of the specimen was close to the measured temperature, and the simulated temperature of d-150 close to the d-110 inside the specimen was different from the measured temperature to a certain extent. This is due to the influence of peeling and thermal resistance between section steel and concrete surface in the test, and in the actual test, the thermocouple arranged on the interface between section steel and concrete cannot completely contact the section steel, and there will be a small amount of concrete wrapped. In numerical simulation, in order to consider the influence of thermal resistance, it is considered that the temperature between them is the same at the contact surface. The numerical simulation results and measured data of d-110 and d-150 close to the inside of the specimen are small (400°C or below), which has little influence on the concrete strength. On the whole, the numerical simulation is in good agreement with the experimental data. Compared with the moment when the ambient temperature reaches the maximum temperature, each point in the cross section shows hysteresis phenomenon, which is more obvious with the increase of distance from the fire surface.

Figure 2: Temperature field

(a) SRHCC30-10  (b) SRHCC30-5  (c) SRHCC25-0  (d) SRHCC25-2
(e) SRHCC35-0

Figure 2  Temperature field
3. Residual bearing capacity

3.1. Basic Assumptions
(1) Uniform distribution of stress in sections of section steel and reinforcement.
(2) The contribution of concrete beyond a certain depth of section in axial bearing capacity is not considered.
(3) In the limit state, all the sections except the protective layer reach the compressive strength of concrete, and the section steel and reinforcement reach the yield strength.
(4) Without considering the influence of interface slippage between section steel, steel bar and concrete, the deformation coordination among materials is considered.

3.2. Calculation method of normal section bearing capacity
The formula for calculating the normal section bearing capacity of the specimen after fire is shown in Formula (1), Where $N_a = f_a(T)A_a$; $N_c = f_c(T)A_c$, and $f_a(T)$, $f_c(T)$ are the yield strength of section steel and reinforcement after fire [9], and $A_a$ and $A_c$ are the sectional areas of section steel and reinforcement after fire. Since the temperature distribution of concrete section under fire is non-uniform, the axial compressive strength of concrete after fire is also non-uniform. In the limit state, the axial force of concrete section is equation (2).

$$N_u = N_c + N_a + N_s$$

$$N_c = \int_A f_c(x,y)ds$$

$f_c(x,y)$ is the concrete strength distribution function on the section. According to the actual temperature field distribution measured by the test and the finite element analysis results, the temperature field distribution shape on the section is shown in Figure 4(I). As shown in FIG. 4 (II-a), $bc$ is a horizontal section, and $bc//oe=b//h_a$, $ab$ is an arc with a circle center and $ob$ as the radius. As shown in FIG. 6 (II-b), area I is the concrete surrounded by the inner surface of the simplified stirrup, and area II is the concrete surrounded by the core area surrounded by the simplified section steel. Thus, the irregular isotherm is simplified into a circle with $oe$ as the radius, Where $\theta$ is constant, $\theta=\pi/4\tan^{-1}\beta$, to
simplify the irregular isotherm for the straight line oe radius circle, \( oe = l_1 = \frac{h_1 \cos \theta}{2} \sqrt{1 + \beta^2} \), 
\( of = l_2 = \frac{h_2 \cos \theta}{2} \sqrt{1 + \beta^2} \).

(I) Contour of concrete strength

(II) Effective section

Figure 4

Calculation and analysis of normal section bearing capacity

\[
N_c = \int_A f_c(x,y)dx = \pi f_{c,e}(T_{II}) l_2^2 + \frac{\pi}{3} l_2^2 (f_{c,e} - f_{c,c}(T_{II})) + \frac{\pi}{3} f_{c,o}(T_{I})(2l_1^2 + l_1^2 - l_1 l_2) + \frac{\pi}{3} f_{c,o}(T_{II})(l_1^2 - 2l_2^2 + l_1 l_2)
\]  

\( T_I \) and \( T_{II} \) are the highest temperatures in the region I and II respectively; \( f_{c,e}(T_{II}) \) is the corresponding compressive strength of concrete in zone II under \( T_{II} \); \( f_{c,o}(T_I) \) and \( f_{c,o}(T_{II}) \) are the corresponding compressive strength of concrete in zone I under the temperature \( T_I \) and \( T_{II} \). The maximum process temperature of point c and Point d were taken respectively. \( h_b \) is the total height of the section steel, \( b_f \) is the flange width, and \( h_h \) is the distance between the inner edge of the stirrup and the center of the section. Let \( N_e = f_c^e(T) A_c \), where \( A_c \) is the net area of full-section concrete, and \( f_c^e(T) \) is the equivalent compressive strength of cross-section concrete after considering the effect of fire. Equation (4) is as follow. Therefore, according to the superposition principle, the calculation formula of the normal section bearing capacity of the specimen is Formula (5). The calculated bearing capacity \( P_{u,p}(kN) \) was compared with the test bearing capacity \( P_{u,t}(kN) \), as shown in Table 4.

\[
f_c^e(T) = \int_A f_c^e(x,y)dx
\]  

\[
N_e = N_c + N_a + N_s = f_c^e(T) A_c + f_{a}^e(T) A_a + f_s^e(T) A_s
\]  

Table 2 Comparison of measured and predicted residual capacity

| Specimen     | PF30-2 | PF30-5 | PF30-10 | PF30-0-H | PF30-2-H | PF25-0 | PF25-2 | PF35-0 |
|--------------|--------|--------|---------|----------|----------|--------|--------|--------|
| \( P_{u,t} \) (kN) | 4916   | 5168   | 6541    | 3733     | 4351     | 3215   | 2992   | 4231   |
| \( P_{u,p} \) (kN) | 4527   | 4622   | 5915    | 3644     | 4513     | 2826   | 2235   | 4621   |
| \( P_{u,p} / P_{u,t} \) | 0.92   | 0.89   | 0.90    | 0.93     | 1.04     | 0.88   | 0.75   | 1.09   |

As can be seen from Table 2, the average ratio of the calculated values to the measured values is 0.92, the variance is 0.097, and the coefficient of variation is 0.11. Except SRHCC30-2-H and SRHCC35-0, the calculated bearing capacity of the other specimens is less than the actual bearing capacity, and the calculated values are relatively safe. Among them, the specimen SRHCC25-0 and SRHCC25-2 the calculation of bearing capacity and the actual bearing capacity is small, because the two specimen of the specimen size is small, under the same reinforcement, large size specimen stirrup can provide strong constraint of concrete action, and the difference of formula did not consider this role, in addition, the calculation method of simplified in the small size of cross section of concrete specimen, the error in calculation result can also cause the smaller, but the calculated value is less than the test value, are safe and suitable for design.
4. conclusion

(1) The temperature field of the specimen presents a symmetric curved edge square temperature gradient distribution, and the closer to the center of the specimen, the higher the temperature of the corner of the specimen is than that of the middle of the specimen with the same horizontal line. The larger the section steel protective layer is, the lower the temperature inside the specimen is, and the thickness of the concrete protective layer has an obvious effect on the core concrete and section steel. Compared with the moment when the ambient temperature reaches the maximum temperature, each point in the cross section shows hysteresis phenomenon, which is more obvious with the increase of distance from the fire surface.

(2) Based on the consideration of temperature cross-section area damage and strength reduction, a simplified calculation formula of residual axial compression bearing capacity after SRHCC column fire is given by using the limit equilibrium theory. The calculated results are in good agreement with the measured and simulated results.

Reference

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