Study on post-fire performance and residual capacity of steel reinforced high-strength concrete composite columns under eccentric compression

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Abstract. In order to combine the advantages of steel reinforced concrete column, high-strength concrete and precast structures, a new steel reinforced high-strength concrete composite column is presented in this paper. With the aim to explore the post-fire behavior of this composite column under eccentric compression, based on the test results, the commercial finite element software ABAQUS was used to simulate the temperature field and then evaluated that sectional damage. The results indicated that the core temperature of the specimen increases with the increase of the thickness of the protective layer of the section steel, and decreases with the increase of the heating time; the longer the heating time, the more serious the section damage is. Based on the theory of equivalent strength, the reduction coefficient of post-fire strength of concrete was determined. Finally, the calculation method of residual capacity of proposed columns was put forward and verified to be valid.

1. Introduction

The steel reinforced concrete (SRC) structure has been widely used in high-rise and heavy-duty structures due to its higher bearing capacity, better seismic performance and durability[1]. However, the on-site construction of SRC structure involves both the steel structure and the steel concrete structure, so the construction procedure of cast-in-place SRC structure is more complicated. At the same time, precast concrete structures are highly praised for their simple on-site construction procedures and better quality control of structural components[2]. Based on the advantages of reinforced concrete and precast concrete, this paper proposed a new steel reinforced high-strength composite column (SRHCC). The section layout is shown in Fig.1. The column is composed of precast part of high strength concrete and cast-in-place part of normal concrete. The precast part is composed of cross-shaped steel, longitudinal reinforcement, continuous stirrups and high-strength concrete; the cast-in-place part is composed of normal concrete. Therefore, it is only necessary to assemble the high-strength prefabricated shell at the construction site and pour the internal normal concrete to form a complete partially prefabricated steel concrete column, which not only effectively simplifies the construction process of the traditional steel concrete column, but also the internal cast-in-place concrete can be combined with the floor slab. The reinforced concrete beam is poured at the same time, which improves the structural integrity while reducing the amount of high-strength concrete to save cost.

Fire can cause varying degrees of damage to structural members. At present, domestic and foreign scholars have carried out more research on the performance of steel reinforced concrete columns under high temperature fire. In order to study the post-fire performance of this kind of partially prefabricated
steel concrete column under eccentric compression based on the existing test results[3], the finite element analysis software ABAQUS was used to simulate the temperature field of this new partially prefabricated steel concrete column under standard fire. The influence of the strength of cast-in-place concrete, the spacing of the stirrups and the size of the cross-section on the temperature field distribution of the specimens was studied in order to provide a reference for engineering design. The basic information of the test specimens in the literature[3] is shown in Table 1.

| Specimen ID | Steel ratio | Outer concrete | Core concrete | Eccentricity ratio | Temperature rising time |
|-------------|-------------|----------------|---------------|-------------------|------------------------|
| SRHCC-1     | 5.16%       | RPC100         | 0             | 0.2               | 120min                 |
| SRHCC-2     | 5.16%       | RPC100         | 0             | 0.4               | 120min                 |
| SRHCC-3     | 5.16%       | RPC100         | 0             | 0.6               | 120min                 |
| SRHCC-4     | 5.16%       | RPC100         | C30           | 0.2               | 120min                 |
| SRHCC-5     | 5.16%       | RPC100         | C30           | 0.2               | 150min                 |
| SRHCC-6     | 5.16%       | RPC100         | RPC100        | 0.2               | 120min                 |

2. Simulation of temperature field

2.1. Thermal properties of materials
In this paper, the thermal properties of steel adopt the values recommended in Eurocode 3[4] and Eurocode 4[5], the thermal properties of normal concrete adopt the values recommended in Eurocode 4[5], and the thermal conductivity and specific heat capacity of RPC adopt the values recommended by Zheng et al.[6]. The boundary condition of the specimen is shown in Fig. 2. The specific values of thermal parameters are shown in Table 2.
Table 2: Thermal properties of steel and concrete

| Thermal properties | Steel | Concrete |
|--------------------|-------|----------|
| of materials       |       |          |
| Thermal expansion (m/(m·°C)) | $a = \begin{cases} 0.8 \times 10^{-4} & (T-20) \\ +1.2 \times 10^{-3} & 20°C \leq T \leq 600°C \\ 0 & 750°C < T \leq 600°C \\ 2.0 \times 10^{-3} & 860°C < T \leq 120°C \end{cases}$ | $\Delta h/l = \begin{cases} 2.3 \times 10^{-11}T^3 \\ +9 \times 10^{-4}T^2-1.8 \times 10^{-4} \\ 20°C \leq T \leq 700°C \end{cases}$ |
| Thermal conductivity (W/(m·°C)) | $\lambda_s = 45$ | $\lambda_c = 1.44 + 1.85 \exp \left( \frac{-T}{242.95} \right)$ |
| Specific heat (J/(kg·°C)) | $c_s = 600$ | $c_c = \begin{cases} 950 & 20°C \leq T \leq 100°C \\ 950 + (T-100) & 100°C < T \leq 30°C \\ \frac{1150 + (T-300)}{2} & 300°C < T \leq 60°C \\ 1300 & 600°C < T \leq 90°C \end{cases}$ |

2.2. Basic Assumptions

The commercial FEA program ABAQUS software was used to simulate the fire temperature rise of the specimen column in this paper. In order to facilitate the fire simulation calculation, the following assumptions were made based on the test results: 1) There is no bond slip between the steel shape and concrete in each specimen; 2) There is no heat source inside the specimen; 3) Rebar, steel shape and concrete are all isotropic materials; 4) The influence of the internal sealing plate is not counted; 5) The contact thermal resistance between the materials is not counted.

2.3. Analysis of modelling and calculation results

DC3D8 (8-node 3D solid element) thermal analysis element was adopted for both steel and concrete modelling; DC1D2 (2-node bar element) thermal analysis element was adopted for modelling longitudinal reinforcement and stirrups. The specimen was placed in an indoor environment before being exposed to fire, with initial temperature 20°C. The comparison between the measured temperature curve and the simulated temperature curve of each specimen is shown in Fig. 3; the temperature cloud of the midspan section and the temperature contour of each specimen are shown in Fig. 4 and Fig. 5 respectively. As shown in Fig. 3, the internal temperature of the specimen calculated by simulation analysis is in good agreement with the overall trend under actual fire conditions of the test. It can be concluded from Fig. 4 that except for the specimen SRHCC-5, which is the cloud image at 150 min, the rest of the specimens are all at 120 min. Due to the fire on all sides of the specimen, the temperature field of the entire cross-section is biaxially symmetrical, and the outer shell is in a rectangular shape with rounded corners, which tends to be more circular as it goes inward. Due to the good thermal conductivity of the section steel, the temperature field at the core is plum shaped, and the petal fullness of the core filled RPC100 is higher than that of the core filled C30. The section temperature gradually decreases from the surface to the inside, the corner temperature is seriously damaged, and the core concrete temperature is relatively low. The closer to the edge of the section, the greater the temperature change variation, and the deeper into the specimen, the smaller the temperature variation. As shown in Fig. 5, after the close of fire source furnace, the temperature continues to rise for a period of time. The deeper the section depth is, the slower the cooling time of the measuring point begins. The historical maximum temperature of cross section decreases with the increase of cross section depth, and increases with the increase of fire time at the same node.
3. Residual bearing capacity

In order to simplify the calculation process, the original section of the specimen was simplified and the transverse flange of the cross section was equivalent to the longitudinal section web. Meanwhile, the octagonal core was equivalent to the rectangular core according to the equivalent principles of area and inertia moment. The equivalent section is shown in Fig. 6.

\[
t_w' = t_w + \frac{0.5 \sum A_{sw}}{h_w}
\]

Where: \( t_w' \) is the equivalent thickness of section steel web, \( \sum A_{sw} \) is the total area of transverse section steel flange; \( t_w \) is the thickness of section steel web.

3.1. Conversion coefficient of post-fire concrete strength

The axial force of the concrete section can be expressed as (2); for the convenience of calculation, the actual iso-strength line of the cross-section is approximately equivalent to the iso-intensity line form shown in Fig. 7, where 3, 4, and 5 are the three areas divided by the shell concrete, and 1, 2 are the two
areas divided by the core concrete. The inner edge of the stirrup is assumed to be the damage control point here. The post-fire strength of the concrete cover is ignored because it is severely lost. The section concrete of the specimen is composed of shell concrete and core concrete. The axial force after the equivalent section strength can be expressed as (3) and (4):

\[
N_{c} = \iint_{A} f_{c}(x, y) \, dx \, dy
\]

\[
N_{c, in} = \iint_{A} f_{c, in}(x, y) \, dx \, dy \\
\approx f_{c, 1} A_{1} + f_{c, 2} A_{2}
\]

\[
N_{c, out} = \iint_{A} f_{c, out}(x, y) \, dx \, dy \\
\approx f_{c, 3} A_{3} + f_{c, 4} A_{4} + f_{c, 5} A_{5}
\]

Where: \(f_{c, 1}, f_{c, 2}, f_{c, 3}, f_{c, 4}, f_{c, 5}\) are the concrete strength values corresponding to the highest historical average temperature in the core concrete area 1, 2 and the shell concrete area 3, 4, and 5 respectively; \(A_{1}, A_{2}, A_{3}, A_{4}, A_{5}\) are the net cross-sectional area of core concrete 1, 2 and shell concrete 3, 4, and 5 respectively. According to the principle of load equivalence, the load is further equivalent as follows:

(Assume that: \(f'_{c, in} = k_{c, in} f_{c, in}\))

\[
N'_{c, in} \approx f_{c, 1} b_{1} h_{1} \left(-t_{w} \, h_{1}\right) + f_{c, 2} \left(b_{2} h_{2} - 2 A_{w} - A_{af} + t_{w} \, h_{1}\right)
\]

\[
k_{c, in} = \frac{f_{c, 1} \left(b_{1} h_{1} - t_{w} \, h_{1}\right) + f_{c, 2} \left(b_{2} h_{2} - 2 A_{w} - A_{af} + t_{w} \, h_{1}\right)}{f_{c, in} A_{c, in}}
\]

In the formula (5) and (6): \(f'_{c, in}\) is the post-fire equivalent compressive strength of the core concrete; \(f_{c, in}\) is the compressive strength of the core concrete at normal temperature; \(k_{c, in}\) is the core concrete strength conversion coefficient; \(A_{wy}, A_{af}\) are the area of the section steel flange and web respectively; \(A_{c, in}\) is the net sectional area of core concrete.

\[
N_{c, out} = f_{c, 3} \left(b_{3} h_{3} - b_{3} h_{1} - 4 A_{s}\right) + f_{c, 4} \left(b_{4} h_{4} - b_{4} h_{1}\right) + f_{c, 5} \left(b_{5} h_{5} - b_{5} h_{3}\right)
\]

\[
k_{c, out} = \frac{f_{c, 3} \left(b_{3} h_{3} - b_{3} h_{1} - 4 A_{s}\right) + f_{c, 4} \left(b_{4} h_{4} - b_{4} h_{1}\right) + f_{c, 5} \left(b_{5} h_{5} - b_{5} h_{3}\right)}{f_{c, out} A_{c, out}}
\]

Where: assume that \(f'_{c, out} = k_{c, out} f_{c, out}\), \(f'_{c, out}\) is the post-fire equivalent compressive strength of shell concrete; \(f_{c, out}\) is the compressive strength of shell concrete at normal temperature; \(A_{s}\) is the area of a single longitudinal reinforcement; \(k_{c, out}\) is the conversion coefficient of shell concrete strength; \(A_{c, out}\) is the net cross-sectional area of shell concrete after removing the concrete cover. Combined with the above temperature field analysis, the specific calculation results and are shown in Table 3.

| Specimen ID | \(k_{c, in}\) | \(k_{c, out}\) |
|-------------|----------------|----------------|

5
3.2. Calculation of post-fire normal section bearing capacity of SRHCC column under eccentric compression

In the calculation of the post-fire normal section eccentric bearing capacity of specimens by referring to existing codes [7], the conversion coefficient of post-fire concrete strength proposed above should be introduced in the calculation of the normal section post-fire eccentric bearing capacity of SRHCC. The results of the comparison between the calculated value and the test value of the post-fire residual bearing capacity of the specimen under eccentric compression load are shown in Table 4. It can be seen from the table that the calculated values are in good agreement with the experimental values.

| Specimen ID | Calculated value/kN | Test value/kN | Calculated value/Test value |
|-------------|---------------------|--------------|----------------------------|
| SRHCC-1     | 2230.1              | 2378.8       | 0.97                       |
| SRHCC-2     | 1478.5              | 1400.3       | 1.05                       |
| SRHCC-3     | 961.5               | 949.7        | 1.01                       |
| SRHCC-4     | 2601.7              | 2873.0       | 0.91                       |
| SRHCC-5     | 2311.3              | 2520.0       | 0.92                       |
| SRHCC-6     | 3304.1              | 3060.3       | 1.08                       |

4. Conclusions

(1) The finite element analysis software ABAQUS can better simulate the cross-sectional temperature field distribution of SRHCC specimens under fire; the cross-sectional temperature gradually decreases from the surface to the inside, the corner temperature damage is serious, and the core concrete temperature is relatively low. The closer to the edge of the section, the greater the temperature change range, and the deeper the inside of the specimen, the smaller the temperature change range. Throughout the fire temperature rise and fall process, the concrete temperature of the core part always maintains a low level, indicating that the shell part can play a better role in protecting the core part under fire.

(2) The post-fire conversion coefficient of the concrete strength of SRHCC specimens was proposed on the basis of the principles of strength equivalence. According to the limit equilibrium theory and considering the post-fire reduction of the cross-section and the conversion of concrete strength, a calculation method for the post-fire normal sectional bearing capacity under eccentric compression of SRHCC column was proposed. The calculated results were in good agreement with the test value, and the proposed method could better calculate the post-fire normal sectional bearing capacity under eccentric compression of SRHCC column.

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