Numerical simulation on the seismic performance of post-fire steel concrete frame joint bonded with steel

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Abstract. Steel was used to reinforce the joint of the frame after fire, and the joint was subjected to low cyclic loading. The seismic behaviors including hysteretic curve, skeleton curve, energy dissipation and stress distribution were analyzed using the finite element method. The results show that the bearing capacity in the elastic stage is almost unchanged comparing with the non-reinforced joint, but the ultimate bearing capacity, energy dissipation capacity and ductility are all improved in different degrees. However, the strengthening effect is not unlimited with the increase of the thickness of bonded steel. The thickness of bonded steel should not exceed 6 mm considering the ultimate bearing capacity, energy dissipation capacity and stress distribution of the structure. The area of high stress zone increases with the increase of axial compression ratio.

1. Introduction

Fire-causing factors are increasing with the development and progress of economy and technology, the probability of fire occurrence and the difficulty of prevention and control are increasing. The material performance of reinforced concrete structure is deteriorated, and its bearing capacity, stiffness and strength index decrease significantly after fire. Most reinforced concrete structures still have certain bearing capacity and can still be used after reinforcement. It can ensure green environmental protection by strengthening and repairing the fire-damaged buildings to make them continue to meet the requirements of use [1]. The working performance of damaged structure is improved by strengthening so that it has certain seismic performance and durability. The joint of frame structure plays an important role in transferring and distributing internal forces, which is very important to ensure the integrity of the structure. Earthquake damage studies show that the joint area is the most vulnerable part of the frame structure. At present, scholars have done some experimental researches on the residual bearing capacity and seismic performance of reinforced concrete beams, columns, shear walls and other components after fire. Some progress has been made in reinforced concrete structures at room temperature [5-9]. However, the limitations of experimental facilities and methods limit the research on structural performance. The reinforcement method of bonded steel plate is simple, and the space occupied is small. It can effectively restrain the corrosion of steel bars and the development of internal concrete cracks [10-14]. It is suitable for the reinforcement of joints. Therefore, it is of great theoretical significance to evaluate and appraise the mechanical properties of reinforced concrete beam-column joints after fire, and put forward reasonable repair and reinforcement schemes for damaged structures to ensure the safety, applicability and seismic performance of the repaired structures.

This paper uses finite element method to simulate the seismic behavior of joints strengthened by steel plates after fire based on the theory of reinforced concrete frame mechanics model. The seismic performance of joints under different reinforcement conditions are evaluated and analyzed.

2. Properties of specimen
The dimensions of reinforced concrete frame joint and cross-sectional dimensions of beam and column are shown in Table 1. The elastic modulus and yield strength of steel bar with diameter 20mm are $1.99 \times 10^5$ MPa and 484 MPa, for diameter 16mm are $2.05 \times 10^5$ MPa and 452.5 MPa respectively. The compressive strength of concrete cube is 31.9 MPa, and the elastic modulus is 3300 MPa. The fire time is 60 minutes, and the four sides are fired.

| Table 1. Reinforcement of joint |
|---------------------------------|
| Strength grade of concrete      | C30   |
| longitudinal steel bar of beam  | 6C16  |
| longitudinal steel bar of column| 6C20  |
| hoop steel bar of beam          | A8@100|
| hoop steel bar of column        | A8@100|
| hoop steel bar in core area     | A8@50 |
| Cross-sectional area of beam    | 200×300|
| (mm·mm)                         |       |
| Cross-sectional area of column  | 300×300|
| (mm·mm)                         |       |
| Cover thickness (mm)            | 30    |
| axial compression ratio         | 0.3   |

3. Simulation of temperature field

A nonlinear finite element simulation of the transient temperature field of the beam is established [15-17] according to the basic principle of heat transfer, and a three-dimensional reinforced concrete frame joint model is established. Results show that the contribution of steel bars to the whole temperature field is not obvious. Therefore, the influence of steel bars on the temperature field of the model is not considered in this paper. The concrete temperature is approximately considered as the temperature of the steel bars.

Thermal parameters of materials including density, specific heat capacity, thermal conductivity and expansion coefficient of steel bar and concrete are determined by the relevant provisions and formulas of European code [18] when simulating temperature field.

The beam and column are all fired on four sides. The temperature amplitude curve is set according to the IS0834 standard fire heating and cooling curve proposed by ISO [19]. The initial temperature is defined as 20°C, the convective heat transfer coefficient is 25 W/(m²·°C), the comprehensive radiation coefficient is 0.7 and the absolute zero is -273°C, Boltzmann constant is $5.67 \times 10^{-8}$ W/m²·K⁴.

The maximum temperature at different positions of the joints after fire are calculated.

4. Reinforcement method

Steel plates are used to reinforce the joints after fire. In this paper, the joints are bonded and strengthened by four-sided wrapping with length 1300mm. Steel plate thickness with 2, 4, 6 and 8mm were selected for analysis. The tensile and compressive strength of the steel plate is 310 MPa, the bending strength is 310 MPa, the shear strength is 180 MPa, the elastic modulus is 206 GPa, and the Poisson's ratio is 0.3. The reinforcement method and size are shown in Figure 3. It is assumed that there is no peel off between the steel plate and the concrete surface.

5. Aseismic behavior of reinforced joints after fire

5.1. Constitutive relationship of materials after fire

Researchers have proved that the mechanical properties of steels are basically restored after high temperature. The yield stress, ultimate stress, elastic modulus, Poisson's ratio and stress-strain of steels are almost the same as those at normal temperature. Therefore, the properties of steel bar after fire are
defined as normal temperature in this paper. The uniaxial hysteresis constitutive relation of reinforcement is selected.

The maximum temperature of each part is different after fire, and the elastic stage of concrete is defined through Young's modulus E and Poisson's ratio. Poisson's ratio is 0.2 which is the recommended value at room temperature because it does not vary with temperature. Young's modulus E at room temperature is obtained by linear interpolation according to the specification [19]. The relationship between temperature and secant modulus of concrete after high temperature is as follows:

\[
\frac{E_{c,Tm}}{E_c} = \begin{cases} 
1.027 - 1.335 \left( \frac{T_m}{1000} \right) & 0 \degree C \leq T_m \leq 200 \degree C \\
1.335 - 3.371 \left( \frac{T_m}{1000} \right) + 2.382 \left( \frac{T_m}{1000} \right)^2 & 200 \degree C < T_m \leq 600 \degree C 
\end{cases}
\]  

(1)

\(E_{c,Tm}, E_c\) are modulus of elasticity of concrete after maximum temperature and normal temperature respectively.

The plastic properties of concrete is analyzed according to plastic potential energy equation and yield surface equation. Expansion angle is 38, flow eccentricity is 0.1, the ratio of biaxial isobaric yield surface equation. The plastic properties of concrete are defined as normal temperature in this paper. The uniaxial hysteretic constitutive relation of reinforcement is selected.

\[
\frac{\sigma}{f_{ch,Tm}} = \begin{cases} 
2x - x^2 & 0 < x \leq 1 \\
115(x-1) & 1 < x \leq \frac{\varepsilon_{0,Tm}}{\varepsilon_{0,Tm}} \\
1 + 5.04 \times 10^{-3} T_m & 1 < x \leq \frac{\varepsilon_{0,Tm}}{\varepsilon_{0,Tm}} 
\end{cases}
\]  

(2)

Which,

\[\varepsilon_0, \varepsilon_u\] are the strain and ultimate strain of concrete at room temperature respectively; \(\varepsilon_{0,Tm}, \varepsilon_{u,Tm}\) are the strain and ultimate strain of concrete after high temperature.

The compressive strength of concrete after high temperature is [12-14]:

\[
f_{cu,Tm} = \begin{cases} 
1 & 0 \degree C \leq T_m \leq 200 \degree C \\
0.0015(200 - T_m) + 1.0 & 200 \degree C < T_m \leq 500 \degree C \\
0.003(600 - T_m) + 0.25 & 500 \degree C < T_m \leq 600 \degree C \\
7.5 \times 10^{-4}(600 - T_m) + 0.25 & 600 \degree C < T_m \leq 800 \degree C 
\end{cases}
\]  

(3)

Which, \(f_{cu,Tm}\) — compressive strength of concrete after high temperature Tm; \(f_{cu}\) — compressive strength of concrete at room temperature.

5.2. Boundary condition and loading mode

Fixed hinges are used at the top of the column to restrain all freedom degrees except U2 and UR1, and all freedom degrees except UR1 are restrained at the bottom of the column. Low cyclic repeated loads are applied at 150 mm from the left and right ends of beams.

The mode of interface contact is: the steel bar and the concrete is embedded constraint, and the bond slip between the steel bar and the concrete is neglected.

5.3. Results and analysis

5.3.1. Hysteretic curve. Hysteretic curve is the load-deformation curve of the structure under repeated loads. It reflects the deformation capacity, stiffness degradation and energy dissipation capacity of the structure under reciprocating loads. It is the basis for determining the restoring force model of the structure and analyzing the non-linear seismic response. The hysteretic curves of structure bonded with different thickness of steel plate show in figure 1.
It can be seen that the hysteretic curves of the strengthened joints can still be clearly distinguished from the elastic, elastic-plastic and failure stages. The different hysteretic curves almost coincide with each other in the elastic stage, and the bearing capacity is improved to a small extent. At this time, the high-strength properties of steel are not effectively utilized. Compared with unreinforced joints, the ultimate bearing capacity and failure displacement of the joints have been greatly improved, which shows that the seismic performance of the bonded joints has been significantly improved. The hysteretic curves after bonded is more plump from a slightly flat diamond to a more full shuttle shape. The hysteretic loop area of the joints with different thickness of steel plate is much larger than that of the unbonded joints in the loading cycle of the same displacement level, which indicates that the energy consumption of the bonded joints has been improved. However, the seismic performance of the joints will not increase with the increase of the thickness of the steel plate.

Figure 1. Hysteretic curves bonded with different thickness of steel

5.3.2. Skeleton curves. The skeleton curve is the envelope of the load-displacement hysteresis curve, which reflects the relationship between cracking load, ultimate bearing capacity, stress and deformation. It is the main basis for seismic performance analysis. It can be seen that the bearing capacity of the bonded joints at the elastic stage is almost the same as that of the unbonded joint from the skeleton curves of the joints bonded with steel plates of different thickness in figure 2. The bond effect is obvious in the elastic-plastic stage, the ultimate bearing capacity, failure load, failure displacement have been greatly improved. The bearing capacity of the joint gradually increases with the increase of the thickness of the bonded steel plate, and tends to be saturated. The descending section of the skeleton curve is gentle when the steel thickness is 2 mm and 4 mm, where ductility is good. Then the descending section begins to become steep with the increase of the steel plate thickness, and the bearing capacity of the joint decreases fastest when the steel plate thickness is 8 mm.
5.3.3. **Bearing capacity.** The yield load, ultimate load and failure load of the bonded joints have been increased to different degrees. The improvement of ultimate bearing capacity represents the seismic effect of the bonded joints because the ultimate load represents the actual bearing capacity of the joints. When the thickness.

| Thickness (mm) | Yield Load | Ultimate Load | Damage Load | Growth Rate of Ultimate Load |
|---------------|------------|---------------|-------------|------------------------------|
| 0mm           | 35011.8    | 46682.4       | 39680.04    | -                            |
| 2mm           | 44822.7    | 58763.6       | 50799.06    | 26%                          |
| 4mm           | 46434.6    | 61912.8       | 52625.88    | 33%                          |
| 6mm           | 49042.43   | 65389.9       | 55581.42    | 40%                          |
| 8mm           | 50892.38   | 67856.5       | 57678.03    | 45%                          |

of bonded steel plate is 2 mm, 4 mm, 6 mm and 8 mm, the ultimate bearing capacity of the joints increases by 26%, 33%, 40% and 45% compared with that of the unbonded joints, indicating that the ultimate bearing capacity increases gradually with the increase of the thickness, but the degree of improvement decreases gradually.

5.3.4. **Energy dissipation.** Energy dissipation capacity is the ability of a structure or component to absorb energy due to irreversible deformation under repeated loads. It is an important parameter to quantify the seismic performance of a ductile structure. In this paper, the viscous damping coefficient he is used to analyze the energy dissipation capacity of a structure. Table 3 shows the energy dissipation of structures bonded with different thickness of steel plate. It can be seen from the table that the viscous damping coefficient of joints increases in varying degrees with the increase of thickness of bonded steel plates. The viscous damping coefficient increases by 19.1%, 25.6%, 22.8% and 13.8% respectively when the bonded thickness is 2 mm, 4 mm, 6 mm and 8 mm. The energy consumption of joints decreases with the increase of the bonded thickness, which may be due to the uneven distribution of stiffness and brittleness of specimens, and the failure modes change.
Table 3. Energy dissipation with different thickness of steel

| thickness | $h_e$ | growth rate |
|-----------|------|-------------|
| 0mm       | 0.2106 | -           |
| 2mm       | 0.251  | 19.1%       |
| 4mm       | 0.2646 | 25.6%       |
| 6mm       | 0.2587 | 22.8%       |
| 8mm       | 0.2397 | 13.8%       |

5.3.5. Stress distribution. Figure 3 shows the stress distribution of the structure under peak load when the axial compression ratio is 0.3, 0.5, 0.7 and 0.8. It can be seen that the high stress zones of the joints increase with the increase of the axial compression ratio. The high stress value of the specimens is mainly concentrated at the end of the beam and the core of the joint where the beam and column intersect when the axial compression is 0.3 and 0.5. At this time, the failure mainly occurs at the end of the beam which belongs to ductile failure. Most of the stress values of the columns exceed the beam end and the core of the joints when the axial compression ratio reaches 0.7 and 0.8, which indicates that the failure of the joints is mainly the collapse of the columns.
6. Conclusion
The seismic behavior of concrete frame joints bonded with steel plates after fire is numerically simulated through the fire temperature field model and the mechanical model of the structure after fire. The results show that the bearing capacity in the elastic stage is almost unchanged comparing with the non-reinforced joint, but the ultimate bearing capacity, energy dissipation capacity and ductility are all improved in different degrees. However, the strengthening effect is not unlimited with the increase of the thickness of bonded steel. The thickness of bonded steel should not exceed 6 mm considering the ultimate bearing capacity, energy dissipation capacity and stress distribution of the structure. The area of high stress zone increases with the increase of axial compression ratio. The failure mode changes from plastic failure at the end of the beam under low axial compression ratio to column collapse under high axial compression ratio.

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