Seismic Performance of Steel Reinforced Concrete Column-Steel Beam Joints after Exposure to Fire

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Abstract. Low cyclic reversed loading tests of six steel reinforced concrete column–steel beam joints were conducted to study the failure mode, hysteretic character, and shear mechanism of joints that had been exposed to fire. Two test parameters, i.e. the duration of fire exposure and axial compressive ratio were taken into consideration. Test results showed that the failure mode of the joints after exposure to fire was basically the same as that of joints maintained at room temperature. Due to the shear resistance of the steel reinforcement, the maximum loading strength of the joints after fire exposure declined indistinctively, however displacement corresponding to the maximum loading strength increased notably. The rigidity, ductility and energy dissipation of the joints after exposure to fire decreased compared to the joints maintained at room temperature, and the extent of reduction was related to heating time and axial compressive ratio.

1. Introduction
Due to the rapid advance in construction technologies, the steel reinforced concrete (SRC) composite structure system enables new buildings with significant advantages such as excellent seismic resistance capability and high loading capacity. This new construction system is widely used in super high-rise and long-span structures; however, fire hazard is a key risk for such structures. Furthermore, the anti-seismic property of SRC composite structures will be weakened after exposure to fire. The SRC structural system has been studied in Japan, America and Europe for more than 50 years, resulting in several theoretically based design formulas [1]. Numerous international publications address the anti-seismic property of SRC composite structures following fire exposure. Related research results were reported by Huang et al. [2, 3], Yu and Lu [4], Jiang et al. [5], Zheng et al. [6] and Mao et al. [7]. The bearing capacity and rigidity of SRC columns after exposure to fire have been tested by Li et al. [8-10], who found that both these two properties were reduced significantly. The residual load bearing capacity of SRC columns after exposure to fire have been analyzed using MARC and ANSYS finite element software, resulting in proposed calculations for determining the residual bearing capacity. For example, Song et al. [11, 12] studied the mechanical performance of SRC column-SRC beam joints after exposure to fire using laboratory tests and finite element analysis. A practical method for calculating the residual strength and residual stiffness of SRC column-SRC beams was developed after analyzing the influence of heating time ratio, fire-to-load ratio, beam line stiffness ratio, strength of materials, the cross-section steel ratio and other parameters.
In contrast, studies on the anti-seismic property of SRC composite structures after exposure to fire are rare. Thus, the purpose of this paper was to study the anti-seismic property of SRC columns–steel beam joints after exposure to fire. In addition to the anti-seismic property, the failure mode and shear capacity of the joints were studied intensively so that the results can be used in engineering practice. Four SRC column–steel beam joints after exposure to fire and two comparable SRC column–steel beam joints at room temperature (without exposure to fire) were tested under low cyclic reversed loading. The hysteretic character, ductility, and energy dissipation of the joints after exposure to fire were measured and compared to those of joints maintained at room temperature. Likewise, tests were conducted to study the capacity and stiffness of the joints after fire exposure.

2. Experimental Program

Based on current engineering practice, Six SRC column–steel beam joints specimens were designed which were subjected to different durations of fire exposure and different axial compressive ratios. Two of the specimens (JD-1 and JD-4) served as controls and were studied at room temperature; the other four specimens (JD-2, JD-3, JD-5 and JD-6) were studied after exposure to fire. Table 1 shows the experimental variables of the six joints in the tests. The influence of fire exposure duration on the anti-seismic property of the joints can be analyzed by examining JD-1, JD-2, JD-3 or JD-4, JD-5, JD-6. The influence of axial compressive ratio on the anti-seismic property of SRC column–steel beam joints can be analyzed by the examining JD-1/JD-4, JD-2/JD-5 or JD-3/JD-6.

Table 1. Experimental variables.

| Joint number | Duration time exposure to fire | Axial compressive ratio |
|--------------|-------------------------------|------------------------|
| JD-1         | 0                             | 0.174                  |
| JD-2         | 90                            | 0.174                  |
| JD-3         | 150                           | 0.174                  |
| JD-4         | 0                             | 0.087                  |
| JD-5         | 90                            | 0.087                  |
| JD-6         | 150                           | 0.087                  |

2.1. Experimental Design

Half-scale internal joints of the frame structure were chosen for the study (figure 1). All the specimens were cut off at the inflection point of a frame beam or column, and hinged joints were used to restrain the beam and column. The length of each SRC column was 1900 mm with an overall cross-sectional dimension of 300 × 300 mm. Each SRC column consisted of a 150 × 150 × 7 ×10 mm H-shaped steel encased in concrete (figure 1(b)). A single, longitudinal bar with diameter of 14mm was placed at each corner of a column. The length of the steel beam was 1100 mm and consisted of a 300 × 125 × 12 × 16 mm H-shape steel. The beam was grooved-welded to the H-shape steel column. The detail of the welded joint is shown in figure 1(b).

The arrangement of steel reinforced concrete column stirrups is shown in figure 1(a). The stirrups with diameter of 8mm were equally spaced at 80-mm centers throughout the joint core area and the densely hooped area. At a length of 500 mm from the joint core area, the stirrups were equally spaced at 150-mm centers throughout the SRC column. Table 2 contains yield strength and tensile strength of the steel reinforcement. And the 28-day average compressive strength of the concrete is 48.5 MPa.
Table 2. Material characteristics of steel

| Types of steel       | Yield strength (MPa) | Tensile strength (MPa) |
|----------------------|----------------------|------------------------|
| Column               |                      |                        |
| Flange               | 479.0                | 643.7                  |
| Wed                  | 536.7                | 627.5                  |
| Beam                 |                      |                        |
| Flange               | 407.4                | 555.1                  |
| Wed                  | 403.6                | 570.6                  |
| Longitudinal bar     | 14                   | 390.0                  |
| Stirrup              | 8                    | 326.4                  |

(a) Section of specimen joint

(b) Size of stiffening rib at core area and beam

Figure 1. Specimen size (mm).
2.2. Heating Curve

The joint specimens were exposed to fire in the fire laboratory of Ningbo University using the fire furnace with eight boccas produced in Zibo, Shandong. The temperature in the furnace was controlled by eight valves manually to adjust the amount and speed of diesel spray into the furnace. Specimens JD-2 and JD-5 were placed in the furnace for exposure to fire for 90 min. Specimens JD-3 and JD-6 were placed in the furnace for exposure to fire for 150 min. Specimens JD-1 and JD-4 were kept at room temperature. Figure 2 shows the heating curves of the four fire-exposed specimens.

![Heating curves of the fire-exposed specimen joints.](image)

2.3. Testing Apparatus and Testing Process

The hinged joint at the bottom or at the top of column can be constrained by processing hinged equipment. The hinged joint at the beam can be achieved by the MTS dynamic actuator (MTS Systems Corporation, Eden Prairie, USA) that is a hinge assembly itself. The axial pressure was generated by a 1000-kN oil jack on the top of column. The MTS dynamic actuator was used to apply cyclic loading to the steel beam of the specimen. Figure 3 shows the test apparatus.

![Test apparatus](image)

The vertical cyclic loading to the steel beam was generated by a 500-kN MTS dynamic actuator with a maximum displacement of ±250 mm. The load-displacement relationship (hysteretic loop) of the beam was automatically collected by the MTS dynamic actuator. Displacement-controlled method was used for loading, and the loading rate was 1 mm/s. Every degree of displacement (2.5, 5, 7.5, 10, 15, 20, 30, 40, 50, 60 and 70 mm) was loaded three times.
A dial indicator was installed and the data acquisition instrument was connected after the specimen and restrain device were installed. The axial pressure was loaded slowly until the pressure reached the target value. Then the acquisition instrument collected the strain data. After that, the MTS dynamic actuator was started to force the low cyclic reversed loading at the beam base on the loading program which was established earlier. The load-displacement relationship (hysteretic loop) was produced by the MTS dynamic actuator itself. Furthermore, the transformation of the column core, crack propagation, and broken concrete at the joint core area for loading at every level was recorded in detail.

3. Experimental Results and Analysis

3.1. Failure Mode
The failure mode of a joint after exposure to fire was a typical concrete shear failure at the joint core area, which was also the failure mode of the control specimens at room temperature. The failure of joints progressed through several visual stages. There was no change at the joint core area at the preliminary stage of loading. Thereafter, cross cracks appeared (45° from horizontal) at the joint core area after loading for a period of time until the beam displacement reached a certain level. As the load increased, the cross cracks at the joint core area became more and more obvious, and divided the concrete into numerous small prisms. The joint of the core area then developed into two cross cracks that became the major cracks. When the load continued to increase thereafter, the concrete around the stirrup in the core area fell off, exposing the stirrup and signifying joint failure. In technical terms, the failure progress of a joint after exposure to fire can be divided into elastic stage, followed by an elastic-plastic stage (work with cracks) and finally a failure stage. Figure 4 shows the typical failure progress of a joint after exposure to fire.

![Initial cracking](a) Major cracking (b) Failure

**Figure. 4** Failure processes and failure patterns of specimens.

3.2. Analysis of Hysteretic Loop
Figure 5 shows the load-displacement relationships (hysteretic loops) of the test specimens collected by the MTS dynamic actuators in this study. The abscissa is vertical displacement of beam and the ordinate is vertical load of beam.

In contrast to the hysteretic loops of joints at room temperature (figure 5(a) and figure 5(d)), the hysteretic loops of joints after fire exhibited a reduced pinch phenomenon, and the area of each hysteretic loop was less than that of joints at room temperature. This difference illustrated that the anti-seismic property of joints after exposure to fire became weaker. Furthermore, with the increase of exposure duration, the hysteretic loop pinch phenomenon was more obvious, indicating that the anti-seismic property of joints decreased with the increase of fire exposure duration.
(a) JD-1 (without exposure to fire, axial compressive ratio is 0.174)

(b) JD-2 (with 90-min exposure to fire, axial compressive ratio is 0.174)

(c) JD-3 (with 150-min exposure to fire, axial compressive ratio is 0.174)

(d) JD-4 (without exposure to fire, axial compressive ratio is 0.087)

(e) JD-5 (with 90-min exposure to fire, axial compressive ratio is 0.087)

(f) JD-6 (with 150-min exposure to fire, axial compressive ratio is 0.087)

**Figure 5.** Load-displacement relationships (hysteretic loops) of joints.
Pinching of the hysteretic loop of joints under 500 kN axial force was more obvious than that of joints under 250 kN force. Thus, hysteretic loop pinching increased as the axial force increased. Nevertheless, the pinch phenomenon was not very obvious as both 500 kN and 250 kN resulted in a small axial compressive ratio.

Visually, all hysteretic loops of SRC joints after exposure to fire resembled a spindle and inverted “S”. The characteristics of a steel structure joint and a reinforced concrete (RC) joint are combined in an SRC structure joint. A SRC joint possesses high ultimate bearing capacity in the initial period of loading because of the combined resistance of steel and concrete, but deforms significantly at later stages of loading due to the failure of concrete. Therefore, the high ultimate bearing capacity and ductility are maintained in a SRC joint even after exposure to fire. Furthermore, the anti-seismic property of SRC structure after exposure to fire exceeds that of RC structure.

3.3. Analysis of Skeleton Curves

The skeleton curve of a joint is the track of ligature of the loading curve at the beginning and peak point of the cycle load-displacement relationship. The feature points of a restoring force model of a structure can be quantified with a skeleton curve. In addition to identifying the feature points of a restoring force model, a skeleton curve can be used to determine yield load, yield displacement, ultimate load and ultimate displacement, as well as the timing of hysteretic characteristics. All the information obtained from skeleton curve is very important. Figure 6 shows the skeleton curve of every specimen joint.

![Figure 6. Skeleton curves of specimen joints (JD-1–JD-6).](image)

The skeleton curves of all SRC joints, whether exposed to fire or not, were smooth and plump without any obvious inflection point, in contrast to curves for RC joints (not shown). These shape characteristics mean that the ductility of a SRC joint is much better than that of a RC joint. Figure 6 also shows that the skeleton curve changed with the duration of fire exposure. Shear capacity of a joint decreased as the duration of fire exposure increased. Furthermore, the skeleton curves also changed as the axial compressive ratio changed. The ascent stage and the descent stage of the curves became much smoother as the axial compressive ratio decreased, indicating that the deformation and ductility improved as the axial compressive ratio decreased. However, the shear capacity of a joint with a smaller axial compressive ratio is less than that of a joint with a larger axial compressive ratio.

Several feature points can be identified using the skeleton curves. Yield load “$P_y$” corresponded to yield displacement “$\Delta_y$”; ultimate load “$P_{\text{max}}$” corresponded to displacement “$\Delta_{\text{max}}$”; and failure load
“P” corresponded to a valid ultimate displacement “Δu”. Table 3 shows the feature points for all specimens. According to the contrast of ultimate condition, bearing capacity of joint will decline with the duration time exposure to fire increased, but with the existent of stiffening rib at core area, the shear capacity does not decline obviously.

### Table 3. Feature points identified from the skeleton curves of specimen joints (JD-1–JD-6).

| State      | Yield state | Ultimate state | Failure state |
|------------|-------------|----------------|--------------|
| Index      | Py (kN)     | Δy (mm)        | Pmax (kN)    | Δmax (mm)    | Pu (kN) | Δu (mm) |
| JD-1       | 102.0       | 17.8           | 129.9        | 29.8         | 110.4   | 52.1    |
| JD-2       | 100.0       | 22.3           | 128.6        | 37.4         | 109.3   | 57.9    |
| JD-3       | 95.0        | 24.3           | 124.5        | 44.9         | 105.8   | 55.2    |
| JD-4       | 116.0       | 24.6           | 148.5        | 45.3         | 126.2   | 65.8    |
| JD-5       | 114.5       | 25.2           | 137.5        | 40.5         | 116.9   | 62.5    |
| JD-6       | 100.5       | 23.8           | 133.0        | 41.8         | 113.0   | 63.8    |

3.4. Analysis of Joint Ductility

The displacement ductility factor of a joint is defined as \( \mu = \Delta_u / \Delta_y \), in which the ultimate displacement is \( \Delta_u \), and the yield displacement is \( \Delta_y \). Yield displacement “\( \Delta_y \)” is confirmed by the bending-moment method. Ultimate displacement “\( \Delta_u \)” is selected as the displacement that corresponds to 85% ultimate load.

Results in table 4 show that the influence of fire on a frame joint cannot be ignored. The displacement ductility factor decreased as the duration of fire exposure increased, which illustrates that long exposure to fire is harmful for the ductility of the SRC column-steel beam joint. The displacement ductility factor of joints with large axial compressive ratios declined much more obviously than that with smaller axial compressive ratios.

### Table 4. Displacement ductility factor for specimen joints (JD-1–JD-6).

| Joint number | Yield displacement Δy/mm | Ultimate displacement Δu/mm | Displacement ductility factor \( \mu \) |
|--------------|---------------------------|----------------------------|-----------------------------------|
| JD-1         | 17.8                      | 52.1                       | 2.93                              |
| JD-2         | 22.3                      | 57.9                       | 2.60                              |
| JD-3         | 24.3                      | 55.2                       | 2.27                              |
| JD-4         | 24.6                      | 65.8                       | 2.68                              |
| JD-5         | 25.2                      | 62.5                       | 2.48                              |
| JD-6         | 24.8                      | 63.8                       | 2.57                              |

3.5. Analysis of Rigidity Degeneration

Rigidity degeneration is defined as the reduction of rigidity with an increase in cyclic loading when the displacement amplitude remains constant, and is also known as annular rigidity expressed by the following equation:

\[
K_j = \frac{\sum_{i=1}^{n} P_i / \sum_{i=1}^{n} u_i}{P_j / u_j}
\]

where \( K_j \) is annular rigidity when the value of displacement \( \Delta_j / \Delta_y \) is “\( j \)”, \( P_j \) is the load at the peak point while loading cyclically at “\( i \)” time, \( u_j \) is the deformation of peak point while loading cyclically at “\( i \)” time, and \( n \) is the total number of cycles.
Figure 7 shows annular rigidity curve of every specimen joint. As can be seen from figure 7, the influence of fire exposure duration on the rigidity degradation of joints is relatively obvious, and the stiffness of joints at room temperature (JD-1 and JD-4) is higher than that of joints exposed to fire. When the displacement was 20 mm, the rigidity of JD-2 degraded by 23.5% compared to the rigidity of JD-1; likewise, the rigidity of JD-3 degraded by 30% compared to that of JD-1. The rigidity of JD-5 degraded by 6.6% compared to the rigidity of JD-4, and the rigidity of JD-6 degraded by 18.8% compared to that of JD-4. The rigidity of all joints declined during the loading process, which was kept constant in the later period of testing when the concrete of the specimens was in failure. At that stage of concrete failure, the rigidity declined little because the steel maintained structural integrity. Due to the high rigidity of concrete, the rigidity of specimens at room temperature declined faster after concrete failure than that of specimens exposed to fire.

The axial compressive ratio had a large influence on rigidity. Compared to specimens with a low axial compressive ratio, the rigidity of specimens with a larger axial compressive ratio was higher at the early stage of loading, but decreased quickly as loading progressed. Figure 8 shows that the rigidity of JD-1 decreased fastest and that other specimen with small axial compressive ratio decreased smoothly.

Figure 8. Energy dissipation curves of specimen joints (JD-1–JD-6).
3.6. Analysis of Energy Dissipation

Energy dissipation is an important basis of a structure’s anti-seismic property. Assuming the structural strength is sufficiently safe, a structure can absorb a large amount of energy from an earthquake without major damage through energy dissipation. Energy dissipation is measured by the equivalent damping ratio $h_e$, which can be calculated as $h_e = A/(2\pi F\Delta)$ in which $A$ is the area within the hysteretic loop.

Table 5 shows the equivalent damping ratio of every joint. The equivalent damping ratios of joints that were not exposed to fire (JD-1 and JD-4) were similar to those of joints that were exposed to fire. In fact, the energy dissipation ratio of joints decreased little in response to fire exposure and remained at favorable values. The energy dissipation of joints with larger axial compression ratios was greater than that of joint with lower ratios (figure 8). The equivalent damping ratio of JD-1 was greater than that of JD-4. The range in the damping ratios among samples was larger in the earlier stage of testing than in the later stage. The variation in the damping ratios of JD-2 and JD-5 was the same as JD-1, and the damping ratio variation of JD-3 and JD-6 was the same as that of JD-4. Because concrete in the earlier stage of testing was in the elastic stage, the larger axial pressure resulted in better energy dissipation. The energy dissipation capacity declined quickly as the concrete reached its yield point in the later stage of testing; thus, the larger the axial compression ratio was, the faster energy dissipation declined. The energy dissipation in the later stage of testing was supported by the stiffening rib in the core area.

Table 5. Equivalent damping ratio of specimen joints (JD-1–JD-6).

| Displacement | 20 mm | 30 mm | 40mm | 50mm | 60mm | 70mm |
|--------------|-------|-------|------|------|------|------|
| JD-1         | 0.077 | 0.114 | 0.165| 0.204| 0.252| 0.301|
| JD-2         | 0.099 | 0.125 | 0.182| 0.244| 0.310| 0.345|
| JD-3         | 0.098 | 0.123 | 0.146| 0.214| 0.276|--|
| JD-4         | 0.051 | 0.092 | 0.110| 0.160| 0.218|--|
| JD-5         | 0.059 | 0.086 | 0.121| 0.166| 0.220|0.276|
| JD-6         | 0.080 | 0.119 | 0.171| 0.231| 0.280| 0.325|

4. Conclusion

Six SRC column-steel beam joints were tested under low cyclic reversed loading to study the seismic performance of the joints after exposure to fire. Four joints were exposed to fire and two joints were maintained at room temperature. Based on the experimental results, the following conclusion can be drawn within the scope of this study:

1. The failure mode and failure process of SRC column-steel beam joints after exposure to fire is similar to that of joints maintained at room temperature.

2. The area enclosed within hysteretic loops, the deformation and the ductility of joint specimens are reduced as the result of fire exposure. The ascent stage of the skeleton curve for a joint exposed to fire becomes smoother, but the descent stage of the curve becomes more abrupt. Furthermore, the skeleton curve changes as axial compressive ratio changes. The ascent stage and the descent stage become much smoother, the deformation and ductility improve as the axial compressive ratio decreases.

3. The ultimate bearing capacity of a joint decreases as the fire exposure duration time increases. However, the shear capacity decline is not obvious due to the presence of the stiffening rib in the core area and the residual bearing capacity of joints after fire exposure remains large.

4. The energy dissipation and ductility of joints after exposure to fire become weaker as the fire exposure duration increases. At the same time, the ductility of joints after exposure to fire has a great influence on the axial compressive ratio. The ductility of joints declines much more significantly for the joints that have a large axial compressive ratio than for joints with smaller ratios. As a whole, the ductility and equivalent damping ratio of SRC column-steel beam joints after exposure to fire remains relatively high, indicating excellent seismic performance.
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References
[1] Wang L G and Li L X 2001 Foreign design specifications basal introduction for steel reinforced concrete (SRC) structure Building Structure 31(2) 23-24,35
[2] Huang Z F, Tan K H and Phng G H 2007 Axial restraint effects on the fire resistance of composite columns encasing I-section Journal of Construction Steel Research 63(4) 437-447
[3] Huang Z F, Tan K H, Toh W S, et al. 2008 Fire resistance of composite columns with embedded I-section steel-effects of section size and load level Journal of Constructional Steel Research 64(3) 312-325
[4] Yu J T, Lu Z D and Xie Q 2007 Nonlinear analysis of SRC columns subjected to fire Fire Safety Journal 42(1) 1-10
[5] Jiang D H, Li G Q, Wang S W, et al. 2005 Research on the resistant capacity of SRC columns subjected to axial compression Steel Construction 20(6) 87-91
[6] Zheng Y Q, Han L H and Jing J S 2008 Research on behavior of steel reinforced concrete beams in fire Engineering Mechanics 25(9) 118-125
[7] Mao X Y, Gao W H, Li L L, et al. 2010 Experimental study on fire resistance of steel reinforced concrete columns subjected to 3-side heating Journal of Natural Disasters 19(6) 93-99
[8] Li J H, Tang Y F, Liu M Z, et al. 2011 Experimental study on the mechanical properties of steel reinforced concrete beams after exposure to fire Chinese Civil Engineering Journal 44(4) 84-90
[9] Li J H, Tang Y F and Liu M Z 2012 Experimental study on the mechanical properties of steel reinforced concrete columns after exposure to fire Journal of Building Structures 33(2) 56-63
[10] Li J H, Tang Y F, Liu M Z, et al. 2012 Experimental study on residual load bearing capacity of SRC columns under axial force after exposure to fire Engineering Mechanics 29(S1) 86-91
[11] Song T Y 2012 Research on Port-Fire Performance of Steel Concrete Composite Beam-Column Joints Tsinghua University
[12] Song T Y, Han L H and Tao Z 2015 Structural behavior of SRC beam-to-column joints subjected to simulated fire including cooling phase Journal of Structural Engineering 141(9) 1-12