Experimental assessment of fire-exposed RC beam-column connections with varying reinforcement development lengths subjected to column removal

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Abstract

This paper describes an experimental investigation into the behaviour of RC beam-to-column connections under a column removal scenario induced by fire. A purpose-built hybrid heating furnace is employed to carry out the fire tests on five RC connection assemblages with varying reinforcement development lengths, with and without cooling effects. The thermal response of the RC connection specimens, including the temperature field and the axial force-furnace temperature curves, is firstly described. Subsequently, push-down tests are carried out on the beam-column connections, and the horizontal support reactions are closely monitored using a specially-designed sensor system. Based on the experimental results, the joint vertical load-displacement and bending moment-rotation relationships are presented, together with an account of the failure modes observed. The mechanical behaviour is discussed in detail, including the tying and rotation capacity provided by the connection specimens, with emphasis on the effect of the reinforcement development length as well as the heating regime. The experimental capacity interaction curves are also compared with the theoretical prediction for ambient condition, and employed to carry out a detailed examination of the underlying failure mechanisms. Finally, the findings are used to provide practical recommendations for enhancing the structural robustness of structural configurations of the form considered in this study.

Keywords: reinforced concrete, beam-column connections, reinforcement development length, fire exposure, catenary action, rotation capacity, tying action.

1. Introduction

The detrimental consequences of progressive collapse have been demonstrated in several structural failures including Ronan Point Tower in London in 1968 and the Alfred P. Murrah Federal Building in Oklahoma City in 1995. Research on progressive collapse has gained considerable attention in recent decades, yet existing codified provisions and design guidelines [1-7] are of limited scope and detail. To address these shortcomings and provide improved understanding of the underlying behavioural mechanisms, a number of experimental and analytical studies have been recently carried out, with particular focus on the ultimate progressive collapse response of framed structures and beam-column assemblages at ambient conditions [e.g. 8-14].

After the World Trade Centre (WTC) events in 2001, Usmani et al. [15-16] demonstrated that the collapse of WTC Twin-Towers is attributed to a fire-induced mechanism rather than the initial damage caused by the attack. Although several existing codes and reports incorporate guidance on the design for structural robustness, this is not immediately applicable to fire situations, and a considerable gap therefore exists between research on fire resistance and structural robustness [17]. Therefore, several recent studies have explored possible approaches for assessing the structural resistance to fire-induced collapse [17-20].

In contrast to collapse under ambient conditions, it is recognized that restraint effects can be significantly amplified during fire-induced failure [21-27]. Structural collapse may be

initiated by a stability mechanism as a result of geometry changes caused by thermal expansion effects, particularly when temperatures are not high enough to generate significant reduction in material properties [15]. On the other hand, thermal-induced restraint effects may lead to additional compression and moment, and further influence the force redistribution [21-22]. Consequently, the fire behaviour of restrained members or structural systems has attracted increasing attention in recent years. Elghazouli and Izzuddin [23-24] proposed a new analytical model for lightly reinforced members with axial restraints, which accounted for the compressive arching and tensile catenary stages, bond-slip, yielding, and rupture of steel reinforcement as well as the effects of elevated temperatures. Dwaikat et al. [25] and Nguyen et al. [26] presented a model for the determination of thermal-induced restraint forces. It was shown that both rotational and axial restraints had distinct influences on the fire resistance. Additionally, the axial restraint ratio, eccentricity, biaxial bending, and concrete compressive strength all improved with the increasing normalized restraint forces. Albrifkani et al. [27] presented a dynamic explicit finite element simulation method to follow the whole behaviour of axially and rotationally restrained RC beams until the final failure.

It is widely recognized that the catenary mechanism, namely tying capacities at large deflections, plays a significant role in the resistance to progressive collapse [8-14]. Meanwhile, catenary action is largely determined by the specific detailing employed. He et al. [28] showed that the current requirements for reinforcement development length in GB50010-2010 [29] satisfied progressive collapse-resistance demands through an experimental assessment. Moreover, Yu et al. [9, 10] and Ren et al. [13] pointed out that seismic detailing

reinforcement of beams had no evident advantage on the collapse-resistance and only the area of continuous rebars determined the ultimate capacity in catenary action. However, notwithstanding the above, other investigations drew the opposite conclusions. For example, Qian et al. [14] and Choi et al [30] showed that seismic detailing and increasing stirrup reinforcement ratios can improve the ductility of plastic hinge regions and enhanced the overall collapse-resistance of RC framed structures. Elsayed et al. [31] also suggested that introducing debonding rebars enabled redistribution of high tensile strains, hence enhancing the beam deformation capacity and permitting larger deformations. Similarly, Yu et al. [32] reported beneficial effects from several special detailing techniques including placing an additional reinforcement layer, partially debonding the bottom reinforcing bars.

Overall, the detailing of reinforcement has been shown to play a significant role in the tying and rotation capacities of RC joints at large deflections under ambient conditions. There is however a need to carry out further studies to understand and quantify the effect of detailing arrangements on fire-induced collapse response. To this end, the experimental investigation described in this paper was carried out in order to provide detailed insight into the key behavioural characteristics, with particular emphasis on the influence of reinforcement development lengths and heating regimes, as well as to provide data for the validation and calibration of future numerical and analytical studies. In this study, RC beam-column connection specimens with varying reinforcement development lengths were firstly exposed to fire, with and without cooling, and then subjected to pushdown loads to simulate typical idealized column removal scenarios. The results include temperature-time, horizontal force-

temperature, vertical load-displacement, and horizontal reaction-displacement response, as well as detailed assessment of axial-moment strength interaction and failure modes.

2. Experimental programme

Six one-half scaled RC beam-column connections were designed and tested under combined fire exposure and fire-induced middle-column loss scenario. The connection specimens consisted of two single-bay beams and a middle stub column, as depicted in Fig. 1. The thermally-induced effect on the response of the unloaded beam, with axial restraint but no rotational restraint, was firstly investigated through heating tests. This was then followed by push-down tests to examine the collapse resistance of RC beam-column connections, where vertical displacement was applied at the top of the column up to failure.

2.1 Specimen details

The prototype of test specimens is set as the middle two bays of a multi-bay frame, and a two-compartment fire is assumed to occur at the central two bays on the ground floor, as shown in Fig. 1. Assuming that adequate strength and stiffness are provided by the surrounding structure, the most critical elements of the frame due to the combined effects of fire exposure and gravity load are considered as the middle column and the attached two beams together with the beam-column connection of the middle two-bay substructure. Moreover, since the internal forces and the deflections of the side beams in a two-bay beam-column substructure are anti-symmetric about the beam mid-span after the middle-column removal, the middle section of the side beams is taken as the contraflexure point for beam deflection. Therefore,

only the beam-column connection assembly with two simply-supported half-span beams was considered in this study to examine the response of RC beam-column connections under a column removal induced by fire.

According to the requirements in [ACI 318-14 \[2\]](#), the positive moment reinforcement extended into the support should be 150 mm at least, while the development length for deformed reinforcements in tension, l_d , shall not be less than 300 mm. According to the corresponding requirements in [GB 50010-2010 \[29\]](#), the development length of tensile reinforcements should not be less than 290 mm. After a middle column fails due to fire, the beam bottom reinforcement near the middle joint would experience a stress transition from compression to tension as a result of reversal of the bending moment. Therefore, the development length of the beam bottom reinforcement at the joint zone becomes a critical parameter for the connection resistance to progressive collapse. The six specimens therefore included different lengths of bottom reinforcement in the joint zone in order to examine this effect. The details of the test specimens are given in [Fig. 2 and Table 1](#).

Due to symmetry, only the details for half of the specimen are shown in [Fig. 2\(a\)](#), including geometric sizes and reinforcement distribution of the RC beams and column. The length of half-bay beams (150 mm × 300 mm) was set as 1370 mm, while the height of the column (300 mm × 300 mm) was 1200 mm. Meanwhile, [Fig. 2\(b\)](#) depicts the layout of thermocouples located in the connection zone. As shown in [Fig. 2\(c\)](#), the bottom moment reinforcement extended into the joint with 150 mm length (for Specimen P150), 230 mm length (Specimens N230 and U230), 290 mm length (for Specimens U290 and P290) mm and continuous (for

Specimen Pc). In the specimen reference, the first uppercase letter N, U or P stands for normal condition, under and post fire, respectively; the following number for the first five specimens in Table 1 refers to the development length, and the lowercase letter c for the last specimen represents the connection with continuous longitudinal reinforcement.

Grade HRB400 steel rebar was used for the beam bottom reinforcement. The material properties of reinforcement and concrete for the RC beams and columns in this paper were tested and are given in Table 2.

2.2 Test set-up

Fig. 3 shows the purpose-built hybrid heating furnace, which consists of a stackable electric furnace, gas fire burners and heating control system. The plane heating modules were fabricated and configured to be a U-shaped electrical furnace, as depicted in Fig. 3(a). Both the left and right side segments consist of three upper and three lower electric heating modules, while the bottom side consists of five insulation modules. Each heating module is made of metallic spiral heating elements embedded in foamed ceramic fibre insulation. The furnace rating is 108 kW/380 V/3 phase with a maximum operating temperature of 1200°C and a maximum heating rate of 100°C/min. A liquefied gas fired facility was used together with the electric furnace in order to accelerate the initial heating process. As shown Fig. 3, this facility was assembled by installing two gas fired burners on each side of the lower modules, allowing the introduction of additional energy when the furnace is being heated by electricity. It has been shown that such hybrid method can be used to heat test specimens effectively and in an energy-efficient manner [33].

The test setup as shown in Fig. 4 was used to conduct the push-down tests on the specimens. The beam-column specimen was placed on the self-balanced support frame, and the specimen was pin-supported and horizontally restrained at each end of the half-bay beams. The vertical load was applied, in displacement control, at the top of the column through a reaction beam with two loading actuators. According to the geometry of test set-up, the net span (L_n) of the specimens was set as 2840 mm, and the effective span (L_e) was 3240 mm including the lengths of the two support connectors (400 mm).

A specially-designed horizontal force measurement system was also added at the pin-support structure, as shown in the longitudinal section in Fig. 4. The beam ends with connectors were connected with the support through pins. The knuckle bearing was then connected to a shaft in the horizontal direction with slide rails at the bottom while, at the other end of the shaft, a sensor was fixed on a rigid plate by bolts. Based on this arrangement, when the specimen is subjected to axial compression, the knuckle bearing attempts to slide outwards, and the axial force is transmitted to the inside nut through the shaft. In contrast, it attempts to slide inwards under axial tension and the force is transmitted to the outside nut. Eventually, the axial force is passed to the sensor, and the horizontal force is accurately measured without the influence of shear and moment. It should be noted that the slide rails at the bottom of the shaft were made from stainless steel and contacted with the upper segment by rolling-sliding balls, and thus the friction force is negligible.

2.3 Test procedure and instrumentation

The specimens (except Specimen N230) were uniformly heated by the specially-built hybrid

heating furnace and were then pushed down to failure. The tests for Specimens U230 and U290 included two stages: heating and push-down loading, whereas Specimens P150, P290 and Pc were subjected to three test stages: heating, cooling and push-down loading. For comparison purposes, Specimen N230 was subjected to push-down loading at ambient conditions. The ambient temperature at the start of the heating tests was about 20°C. The specimens were heated by using the hybrid heating method [33], and the heating phase was automatically controlled following the ISO-834 standard fire curve [34]. During the cooling stage, the specimens were cooled down to a safe temperature, and the furnace was then removed in order to effectively cool down the specimens to ambient temperature for the push-down tests. Fig. 5 shows the time history of furnace temperature for all the tests during the heating and cooling stages, together with the ISO-834 standard fire curve. The mean temperature of the furnace was similar for all the test specimens, but the development of measured furnace temperature was below the ISO-834 standard fire curve. After the fire exposure test, a vertical load was applied at the top of the column top stub in displacement control at a rate of 40 mm/min until the bottom reinforcements near the connection zone fractured or pulled out.

The temperatures of the furnace as well as the beams and beam-column connection during the heating and cooling stages were recorded by seven thermocouples, as indicated in Fig. 2(b). Five thermocouples (M1-M5) were embedded inside the RC beam to measure the temperature, and two thermocouples (M6 and M7) were embedded in middle section of the joint to record the temperature development of the upper and bottom anchorage reinforcement extended into

the joint. The horizontal reaction forces of both beam supports were recorded by the special measurement system during the heating, cooling and loading stages. The vertical displacement of the column, as well as the corresponding applied load, was also measured through the actuator during the push-down loading.

3. Experimental results

This section presents the main experimental results obtained from the tests. These include the thermal response of the RC beam under high-temperature exposure as well as the mechanical behaviour of the fire-damaged RC beam-column connections. Two typical representative specimens were selected to illustrate the temperature-time curves, and they are shown in Fig. 6. while Fig. 7 presents horizontal force-temperature curves. The post-fire crack patterns at the beam ends are illustrated in Fig. 8. The overall structural behavior of the connections under push-down loading, in terms of the vertical load-displacement and horizontal reaction-vertical displacement response, is shown in Figs. 9 and 10, respectively. Finally, the failure modes of all specimens are illustrated in Fig. 11.

3.1. Thermal response

The measured temperatures at various depths of beam cross-section (M1-M5) and the joint (M6-M7) are plotted in Fig. 6(a) and (b) for Specimens U290 and Pc, respectively. It is shown that, as expected, the temperatures at the surface layer concrete, especially at the corner of the specimen section, developed faster compared with those of the inner concrete layers, which led to the observed temperature gradient within the specimens. There was also a clear temperature plateau around 100°C, which is attributed to the evaporation of water in concrete,

after which the concrete temperatures gradually increased with the fire exposure time. For Specimen U290, the temperatures recorded at Points M1, M3 and M5 at 120 min were 519°C, 459°C and 543°C, respectively. The temperatures at the middle points (M6 and M7) of anchorage reinforcements extended into the joint were 103°C and 123°C, respectively. For Specimen Pc, the temperatures exhibited similar development trends to those of Specimen U290 during the heating stage. During the cooling stage, the temperatures of the surface layer concrete at Points M1, M3 and M5 began to decrease immediately. However, the temperatures of the inner concrete layers at Points M2 and M4 firstly continued increasing to the peak values at 150 min, then remained at the highest level before finally dropping slowly and steadily. Similarly, the temperatures of the anchorage reinforcements at Points M6 and M7 increased to maximum values at about 240 min and then became stable. The highest temperatures at all measuring points are listed in Table. 3.

The thermal response of the specimens is also reflected in the horizontal support force-furnace temperature curves in Fig. 7(a) and the horizontal force-time curves in Fig. 7(b). It is observed that during the heating stage, all specimens were under compression due to axially restrained thermal expansion. Meanwhile, the horizontal reaction-temperature development was similar for all the specimens with the same beam cross-section, due to the similar heating rates of the fire-exposure tests, as shown in Fig. 5. The horizontal forces below 400°C were very small and could be neglected, but increased sharply and almost linearly when the furnace temperature was above 500°C. During the cooling stage, the horizontal forces decreased more slowly than the increasing rate during heating. The decreasing rate of horizontal force was

relatively small before the furnace temperature reduced to 400°C, whereas the horizontal forces decreased to zero at about 40°C at a higher rate. It is also observed that Specimens P150, P290 and Pc experienced small tension forces when they were cooled down to room temperature. Fig. 7(b) indicates that it took about 4-6 hours for the furnace temperature to be cooled down to room temperature.

The cracking patterns of Specimens P150, P290 and Pc resulting from heating and cooling are shown in Fig. 8. The cracks marked by the red lines spread mainly vertically at the sides of beam ends within the length of one beam height. Some tension cracks started from the top of the beam and propagated downwards, which is related to the thermal bowing due to the non-uniform heating of the cross-section of the restrained specimens. Other cracks also occurred in the beam web due to the axial shortening of the specimens during the cooling phase. However, the actual sizes of these cracks changed during the cooling phase.

3.2 Push-down response

Fig. 9 presents the vertical load-displacement relationship for all the specimens as obtained from the push-down tests. It can be observed in Fig. 9 that the vertical load-displacement curves can be generally divided into the four stages: beam mechanism stage, compressive arch action stage, transient stage and catenary stage. However, it should be noted that the development of vertical force for Specimen P150 was different due to the discontinuous anchorage reinforcement in the joint zone. At the beam mechanism stage, the development trends of the vertical load-deflection histories were similar. All the specimens experienced peak load points at the compressive arch action stage. It is also noted that the peak values of

the specimens were almost the same except Specimen U290. This is because U290 experienced relatively lower inner temperature as listed in Table.3 and its material properties degraded less, but horizontal reaction forces were almost the same as shown in Fig. 7. After the first peak points, there was a considerable difference in response, as the vertical forces of Specimens U230 and U290 under high-temperature exposure dropped down immediately, whereas other curves maintained a relatively stable load. At the transient stage, the curves of Specimens U230 and U290 continued to reduce sharply, while the others only slightly decreased. At the catenary action stage, it is obvious that only the curves of Specimens P290 and Pc developed an ascending branch, while the vertical forces of Specimens U230 and U290 stabilized, dropping suddenly for Specimen N230 at failure.

Fig. 10 depicts the horizontal support reaction-vertical displacement curves of the specimens. For Specimen N230, axial compressions (i.e., negative value) grew up with increases of column vertical displacement until 100 mm, and it then reduced to nearly zero when the displacement approached about 180 mm. For Specimens P290 and Pc, the development curves were similar with that of Specimen N230, but they experienced axial tension (i.e., positive value). In addition, Specimen Pc exhibited greater axial tension than Specimen P290. For Specimens U230 and U290, the high-temperature exposure resulted in the initial axial compression value of about 210 kN. Both curves had similar shapes to those of other specimens, but the response was dominated by compression.

For ease of comparison with the critical forces and displacements, Table 3 summarizes the corresponding test results of all specimens in the above sections, where F_p , F_u , δ_p , δ_u represent

the peak force, ultimate force, and the displacements corresponding to the peak force and ultimate force, respectively, noting that $N_{\max}$ represents the maximum axial force. It is found that Specimens P290 and Pc developed the same value of F_p , which was 93% of that of Specimen N230, indicating that two different development lengths had little influence on the peak force in this period. However, the values of F_p for Specimens U230 and U290 were 92% and 119% of Specimen N230, respectively. It is attributed to different level of the horizontal force as show in Fig. 10. Moreover, Specimens U230 and U290 developed an identical value of F_u , which was 39% of that of Specimen N230, while the value of F_u for Specimens P290 and Pc were 122% and 177%, respectively, which was greater than that of Specimen N230. It is indicated that the post-fire condition and the increasing development lengths enhanced the connection ultimate force. Furthermore, the displacements corresponding to the peak force (δ_p) of Specimens U230 and U290 were about half that of Specimen N230, while δ_p of Specimens P290 and Pc were almost the same as that of Specimen N230. Additionally, Specimens U230 and U290 exhibited similar values of δ_u , which were 27% and 33% larger than that of Specimen N230, while δ_u of Specimens P290 and Pc were 13% and 42% higher than that of Specimen N230. It is further showed that the post fire condition and the longer development lengths also improved the connection deformation capacity.

3.3 Failure modes

Due to the application of a vertical force onto the middle joint, the beam-column interface zone was subjected to a combination of positive bending moment and axial force. The ultimate response, including significant concrete cracking, concrete spalling and crushing, and

reinforcement fracture, was mainly concentrated at the beam-column interfaces. Consequently, more attention was given to the cracking pattern and failure modes in these local areas, as shown in Fig. 11.

As shown in Fig. 11(a), when Specimen N230 prior to ultimate failure through reinforcement rupture at the beam-column interface, a few flexure cracks and hairline inclined cracks occurred, and one large crack separated the beam and column, indicating that the plastic hinge area was relatively narrow and near the interface. As observed in Figs. 11(b) and (c), and compared with Specimen N230, a smaller number of wider cracks developed in Specimens U230 and U290, with bottom reinforcement rupturing away from the interface coupled with concrete crushing at the beam upper surface. These typical patterns, especially for Specimen U230, indicated that the plastic hinges were shorter and moved far away from the beam-column interface due to the thermally induced axial compression.

Fig. 11(d) shows the failure mode of Specimen P150. It can be seen that in this case no significant cracking occurred, and beam and column separated entirely as the bottom reinforcement pulled out due to unduly short development length. On the other hand, the cracking patterns and failure modes of Specimens P290 and Pc are presented in Figs. 11(e) and (f). The red and blue marked lines on the surface signify the temperature cracks and loading cracks, respectively, where the even distribution of temperature cracks is observed. Specimens P290 and Pc developed more transverse cracks but fewer inclined cracks, and the bottom reinforcement ruptured away from the interface with the column, coupled with concrete spalling and crushing. The response of these two specimens indicated that not only

have the length and location of plastic hinges extended considerably, but that the beam behaviour was also significantly affected by axial tension.

4. Comparative behaviour

In order to further study the influence of the reinforcement development length and heating regime on tying and rotation capacity, the bending moment–rotation curves of all connections are depicted in Fig.12, and several key response parameters are summarized in Table 3. Discussions focusing on the rotation capacity and failure mechanisms are presented below.

4. 1. Rotation capacity

In order to examine the rotation capacity of each beam-column connection, Eq. (1) is used to calculate the bending moments at the beam-column interface. This illustrates that the bending moment was influenced by the following two aspects: i) increasing vertical force resulting in a larger M ; and ii) presence of beam axial compression, with the $N\text{-}\delta$ (i.e. second-order effect) increasing with displacement and leading to a smaller M .

$$M = R \cdot L_c - N \cdot \delta \quad (1)$$

where M is the bending moment acting at the beam-column joint interface with reference to the geometrical center of beam section; R and N represent the vertical support reaction force and axial compression, respectively; δ denotes the middle joint deflection; and L_c stands for the calculated span length between the beam support and beam-column interface. The connection rotation is also represented by δ/L_c .

The bending moment-rotation curves are depicted in Fig. 12. The maximum moments, ultimate moments and ultimate rotation angles are summarized in Table 3. It is observed that

the ultimate rotational angle of Specimen N230 was the smallest. Compared with Specimen N230, the post-fire specimens (P290 and Pc) exhibited more spread bending-rotation curves except Specimen P150. Specimens P290 and Pc experienced lower maximum moment than Specimen N230, while the ultimate rotations of the former connections were larger. This may be attributed to three main aspects. Firstly, the longer development length played a positive role in rotational capacity under different loading conditions. Secondly, although post-fire materials were damaged to some degree, generally providing positive effects, several studies [35-37] have showed that the deformation capacity of concrete improved due to elevated-temperature exposure. Additionally, various investigations [28, 31, 32] indicated that partial debonding due to fire exposure, can lead to more uniform strain distribution along the rebars, hence enhancing the failure displacement. On the other hand, the on-fire connections (Specimens U230 and U290) exhibited steep responses. Their ultimate rotations were also higher than that of Specimen N230 due to fire exposure, while their maximum bending moments were notably enhanced, which can be explained by thermal compression effects.

4.2. Failure mechanisms

To further investigate the effect of reinforcement development length and heating regime on the failure mechanism, it is necessary to examine the evolution of force state during the entire loading process. The experimental N - M interaction curves of all connections are plotted in Fig. 13(a), where N is the axial force also representing approximately the support horizontal reaction, and M is the bending moment as evaluated in Section 4.1. The N - M capacity curve for ambient cases is derived by using the program Response-2000 [38], which is based on the

Modified Compression Field Theory (MCFT) [39], and introduced in Fig. 13(a) so as to further understand the influence of reinforcement development length and fire scheme on the tying and rotation performance of restrained RC connections. Moreover, the catenary capacity for ambient cases is estimated by Eq. (2) proposed by Rashidian [40] as also shown in Fig. 13(a).

$$F_{catenary} = 0.4 A_{s_{top}} \cdot f_{u_{top}} \quad (2)$$

where $F_{catenary}$ is the catenary capacity, $A_{s_{top}}$ denotes the area of beam top reinforcement; $f_{u_{top}}$ represents the reinforcement ultimate strength. further assessments of the evolution of the force state, failure process, as well as the failure mechanism under different loading conditions was carried out in this section, based on Fig. 13 and the comparison study in the above sections.

According to the different loading states, the evolution path of N - M interaction for the normal connection (Specimen N230) shown in Fig. 13(a) can be divided into the following three segments: bending stage (Path O-A), compression-bending hardening stage (Path A-B), and compression-bending degrading stage (Path B-C). Before Point A, the specimen was subjected to flexural action with low axial force. Due to the existence of axial compression forces, the ultimate bending moment was enhanced from A to B, where Point B is very close to the capacity curve, hence it approaches the ultimate flexural capacity and the end of the compressive arch action. After Point B, the specimen deterioration of flexural capacity occurred, so the path regressed due to second order effects. It is generally observed that Path B-C nearly followed the capacity curve. Moreover, from Points B to C, Specimen N230

experienced crushing of concrete as depicted in Fig. 11(a) as well as hardening of the bottom reinforcement. Finally, Specimen N230 failed to develop catenary action. Compared with tests described in previous studies [8, 9, 10], this can be attributed to the shorter development length of the bottom reinforcement.

For the post-fire specimens (P290 and Pc), the evolution paths of O-A-B-C were similar to that of the normal specimen, but the curves extended to Path C-D, namely the tension-bending stage, as depicted in Fig. 13(a). At Point A, Specimens P290 and Pc showed a lower maximum moment due to the deterioration of material properties. Also, the moment of Specimen Pc was lower than that of Specimen P290 because of different temperature fields as listed in Table. 3. Meanwhile, at Point B, Specimens P290 and Pc exhibited relatively low capacities when compared with the normal specimen. From Points B to C, it is noticed that the Paths B-C of Specimens Pc and P290 increased gradually beyond that of normal specimen. Moreover, only the $N-M$ curves of Specimens P290 and Pc can extend from Point C to D, which indicates the formation of catenary action. Interestingly, Specimen Pc experienced the longer path (C-D) and exceeded the $M-N$ capacity curve, which may play a positive role in strengthening the structure in the post-fire state. Overall, Specimens P290 and Pc showed relatively low capacities in the Path O-A-B, but exhibited better performance in the B-C-D range. This can be attributed not only to the effect of the longer development length, but also to the improvement of concrete deformation capacity after exposure to the high temperatures. This is also contributed to by the uniform strain distribution along the bottom reinforcement bars in the vicinity of the joint interface due to the positive effect of the deterioration of bond

properties caused by heating treatment [1, 12, 23, 39, 40].

However, as Specimen P150 was designed with shorter reinforcement development length, it failed to fully develop flexural action due to the weak anchorage. As observed in Fig. 11(d), no significant cracking occurred, and the bottom reinforcement eventually pulled out before the formation of a plastic hinge. Therefore, the least development length of the positive moment reinforcement extended into the support, based on the detailing requirements of ACI 318-14 [2] (150 mm), cannot prevent the progressive collapse to occur after a two-hour fire according to the corresponding study in this paper. The reinforcement development length hence had a notable effect on the rotation and tying capacity of RC beam column connections. For the heating specimens (U230 and U290), due to thermal axial compression the N - M development began directly from the compression-bending hardening stage (Path E-F), followed by the compression-bending degrading stage (Path F-G), as shown in Fig.13(a). At Point F, it reached the maximum bending moment and axial compression with a smaller mid-displacement than that at Point B for post-fire and normal specimens as observed in Figures 9, 10 and 12, due to the more significant N - δ effect. Additionally, as the different temperature fields (as listed in Table. 3) resulted in different degrees of material degradation and compressive arch capacity, Specimen U230 and U290 approached a different point (F). Following the path of F-G, both specimens started to regress and failed to enter tension-bending stage. To facilitate comparison of the N - M evolution trends of Specimens U230 and U290 with the N - M capacity curve, as depicted in Fig. 13(b), the experimental N - M curves of Specimens U230 and U290 were moved upwards to the coordinate origin by setting the initial

axial compression as zero, and two new translational N - M curves, referred to as U230* and U290* were obtained. It is found that the axial compression increased slowly and Point F was extended beyond the N - M capacity curve, indicating that the thermal axial compression enhanced the bending capacity to a certain extent. Moreover, the path of F-G regressed more quickly when compared to that of other specimens, as the initial axial compression increased the rate of degradation. However, as depicted in Fig. 11(e-f), Specimens U230 and U290 developed shorter plastic hinges away from the beam-column interface due to the thermally-induced axial compression. Therefore, the thermally-induced axial compression would have negatively affected the failure mode as well as the rotation and tying capacity. For the RC connection assemblages under elevated temperatures, it is necessary to change the restraint conditions by using specific construction measures at the end of the beam so as to improve their rotational capacity.

Finally, the above discussions have highlighted the need for further detailed assessment on quantifying the following aspects: i) tying capacity, as governed by the anchoring performance; ii) rotation capacity, as determined by the development of plastic hinges; iii) thermal restraint effects, as influenced by the restraint stiffness of the surrounding structure.

5. Conclusions

This paper provided a detailed description of six tests on RC beam-column connection specimens which were subjected to the removal of the middle column to investigate their progressive collapse performance under different heating regimes. The following main conclusions can be drawn from the test results:

- (1) Compared with the specimen tested at ambient (N230), the typical failure patterns developed for on-fire connections (Specimens U230 and U290) demonstrated that the plastic hinges were shorter and moved significantly away from the beam-column interface due to the thermally-induced axial compression. On the other hand, the typical failure patterns of the post-fire specimens showed more transverse cracks but fewer inclined cracks, indicating that not only have the length and location of plastic hinges extended considerably, but that the beam behaviour was also significantly affected by axial tension.
- (2) It was shown in the tests that the least development length of the positive moment reinforcement which extended into the support based on the detailing requirements of [ACI 318-14](#) (150 mm) cannot prevent the occurrence of progressive collapse after a two-hour fire exposure.
- (3) Compared with the normal specimen (N230), Specimens P290 and Pc showed relatively low capacities in the bending stage and compression-bending hardening stage, but exhibited better performance in the compression-bending degrading stage and also entered the tension-bending stage. This can be attributed to the effect of the longer development length as well as to the improvement of concrete deformation capacity after exposure to high temperatures. Another contributing factor was the more uniform strain distribution along the bottom reinforcement bars in the vicinity of the joint interface due to the positive effect of the deterioration of bond properties caused by heating treatment.
- (4) The on-fire connections showed larger rotation stiffness due to the more significant $N-\delta$ effect induced by thermal axial compression. It also skipped the bending stage and began

from the compression-bending hardening stage, followed by the compression-bending degrading stage, but failed to develop a catenary mechanism.

The findings of this study point to the need for further future studies to quantify the connection performance in terms of tying and rotation capacity as well as the thermal restraint effects due to interaction with the surrounding structure.

Acknowledgements

The authors gratefully acknowledge the financial support of the National Natural Science Foundation projects (Projects No. 5137820 and No. 51408212), the Program for New Century Excellent Talents in University (NCET-11-0123) and the Program for Changjiang Scholars and Innovative Research Team Project (Project No. IRT0619), for the research described in this paper. The authors are also grateful for the support for the experimental work at the Center for Integrated Protection Research of Engineering Structures (CIPRES), Ministry of Education Key Laboratory of Building Safety and Efficiency, at Hunan University.

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Figures:

Fig. 1 Schematic view of beam-column connections under column removal scenario induced by fire

(a) Details of beam-column connections

(b) Layout of located thermocouples

(c) Detailing of beam bottom reinforcements

Fig. 2 Configuration of RC beam-column connections (units: mm)

(a) Schematic representation

(b) General view

Fig. 3 U-shaped hybrid heating furnace

Fig. 4 Push-down test set-up (units: mm)

Fig. 5 Time history of furnace temperature for five specimens together with ISO-834 standard curve

(a) Specimen U290

(b) Specimen Pc

Fig. 6 Typical temperature-time curves of Specimens U290 and Pc during heating and cooling stages

(a) Horizontal force-furnace temperature curves

(b) Horizontal force-time curves

Fig.7 Development of support horizontal force during heating and cooling stages

Fig. 8 Crack pattern of beam ends after heating and cooling stages

Fig. 9 Vertical load-displacement relationships of all specimens under the push-down tests

Fig. 10 Support horizontal reaction-vertical displacement curve of all specimens under the push-down tests

Fig. 11 Final failure modes of all beam-column connections after push-down tests

Fig. 12 Bending moment-rotation curves of all specimens under the push-down tests

(a) Original curves

(b) New translational N-M curves of U230 and U290

Fig. 13 N-M interaction curves of all connections within the comparison of corresponding capacity curve and catenary capacity

Tables :

Table.1 Details of test specimens

Specimen Reference*	L_n (mm)	Reinforcement ratio						Stirrup ratio	Condition	Detailing at Bottom steel	Development length
		Section 1-1 (Fig. 2)		Section 2-2 (Fig. 2)		Section 3-3 (Fig. 2)					
		Top	Bottom	Top	Bottom	Top	Bottom				
N230	2840	2C14 (0.77%)	2C14 (0.77%)	2C14+1C16 (1.28%)	2C14 (0.77%)	4C20 (1.39%)		0.67%	Normal condition	Splice	230
U230	2840	2C14 (0.77%)	2C14 (0.77%)	2C14+1C16 (1.28%)	2C14 (0.77%)	4C20 (1.39%)		0.67%	Under 2h fire	Splice	230
U290	2840	2C14 (0.77%)	2C14 (0.77%)	2C14+1C16 (1.28%)	2C14 (0.77%)	4C20 (1.39%)		0.67%	Under 2h fire	Splice	290
P150	2840	2C14 (0.77%)	2C14 (0.77%)	2C14+1C16 (1.28%)	2C14 (0.77%)	4C20 (1.39%)		0.67%	After 2h fire	Anchorage	150
P290	2840	2C14 (0.77%)	2C14 (0.77%)	2C14+1C16 (1.28%)	2C14 (0.77%)	4C20 (1.39%)		0.67%	After 2h fire	Splice	290
Pc	2840	2C14 (0.77%)	2C14 (0.77%)	2C14+1C16 (1.28%)	2C14 (0.77%)	4C20 (1.39%)		0.67%	After 2h fire	Continuous	—

*Specimen reference: the first letter N, U or P stands for normal condition, under and post fire, respectively, while the number for the first five specimens refers to the anchorage length, and the lower-case letter c for the last specimen represents the connection with continuous longitudinal reinforcements. Grade HRB400 steel bar was used for all beam bottom reinforcement.

Table.2 Material properties for reinforcement and concrete

Material	Item	Measured value			
		C8	C14	C16	C20
Reinforcement	Yield strength (MPa)	588	533	502	501
	Ultimate strength (MPa)	785	649	620	641
	Elongation δ_{10}	15.1%	14.2%	15.3%	15.8%
Concrete	Cube strength (MPa)	47			

Table.3 Summary of test results for the six specimens

Specimen	Measuring point*							F_p (kN)	δ_p (mm)	F_u (kN)	δ_u (mm)	N_m (kN)	M_m (kN·m)	M_u (kN·m)	θ (rad)
	M1	M2	M3	M4	M5	M6	M7								
N230	-	-	-	-	-	-	-	60.5	79	53.8	184	-99.3	52.6	42.4	0.131
U230	615°C	555°C	574°C	340°C	625°C	170°C	177°C	56.0	43	20.9	234	-280.0	56.7	14.9	0.161
U290	519°C	302°C	459°C	295°C	543°C	103°C	123°C	71.7	39	20.6	244	-325.2	67.1	18.6	0.166
P150	420°C	356°C	445°C	288°C	510°C	140°C	142°C	17.0	11	12.7	26	-63.7	13.1	12.1	0.019
P290	385°C	295°C	427°C	293°C	473°C	124°C	173°C	56.8	83	65.5	217	-84.9	50.8	39.5	0.150
Pc	419°C	355°C	555°C	301°C	609°C	117°C	160°C	56.8	74	95.1	262	-116.2	53.7	35.4	0.184

* *Measuring Point*: M1-M7 denote the 2-hour temperatures at the corresponding thermocouple points; F_p , F_u , δ_p , δ_u represent the peak force, ultimate force, the displacements corresponding to peak force and ultimate force, respectively; N_m is the developed maximum axial force; M_m and M_u stand for the maximum moment and ultimate moment, respectively; θ indicates the ultimate rotation angle.