External reinforced concrete beam-column subassemblages in fire

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ABSTRACT: Structural behaviour of reinforced concrete (RC) beam-column subassemblages under fire conditions have always been a place of interest for engineers. However, due to the difficulties associated with conducting fire tests on the RC beam-column subassemblages subjected to service loads, empirical data has been limited. In order to investigate the fire resistant behaviour of the aforementioned subassemblages, a frame consisting of two cantilever beams was constructed. Equal upward and downward service loads were applied at the end of each beam, respectively. Several thermocouples, strain and displacement gauges were installed at different sections. The top faces of the cantilever beams and the upper columns were thermally insulated. The formation of flexural and thermal cracks and the loss of bond strength at elevated temperatures led to a significant drop in fire-resistant behaviour compared to the ambient temperature.

1 INTRODUCTION

Beam-column subassemblages play a significant role in maintaining the serviceability of RC structures. The structural behaviour of this part has been investigated by several researchers. However, there has been little research conducted in fire conditions (Nishiyama 2011). This could be due to the hardships of simulating and conducting fire tests on such subassemblages subjected to specific loadings. In this paper, in order to provide more specific empirical data, two beam-column subassemblages of one statically indeterminate frame were made to be tested in a large scale furnace. Due to the decrease of bond strength and also formation and widening of cracks at elevated temperatures, the fire resistant performance of the specimens drastically decreased. The decrease of bond strength between the reinforcing rebar and concrete at elevated temperatures and the effect of cracks in decreasing structural behaviour of RC beams in fire condition is under investigation by the authors.

The purposes of this research at the first step were measuring (a) internal temperature distribution of beam-column connection, (b) deformation of beam, column, and beam-column connection, (c) crack width before and after the fire test, (d) reinforcement strain at elevated temperatures, and (e) the damage due to fire. In the second step, the test specimens were simulated in a full 3D finite element (FE) model. By proving the mechanical and thermal-dependent characteristics of each material, the FE results were compared to that of the tests; however, the FE modelling and results will be presented elsewhere.

2 EXPERIMENT WORK

2.1 Specimens setup

As it is shown in Figure 1, two RC beam-column sub-assemblages were constructed in scale of 1/3. The cross-sections of beams and columns were 200×250mm and 250×250mm, respectively. The two columns' bases and heads were linked together via two H-shape steel beams (250×250×9×14). Four prestressing steel rods (ϕ26mm, SBPR 1080/1230) were used to connect the H-beams to the columns bases and heads tightly by post-tensioning. The prestressing load for each prestressing steel rod was 141 kN. The details of reinforcement and materials are shown in Table 1 and 2, respectively. Two equivalent service loads of 17.3 kN based on long-term allowable stress capacity design were applied at the left end beam (Push-down side) and right end beam (Pull-up side) downward and upward, respectively.

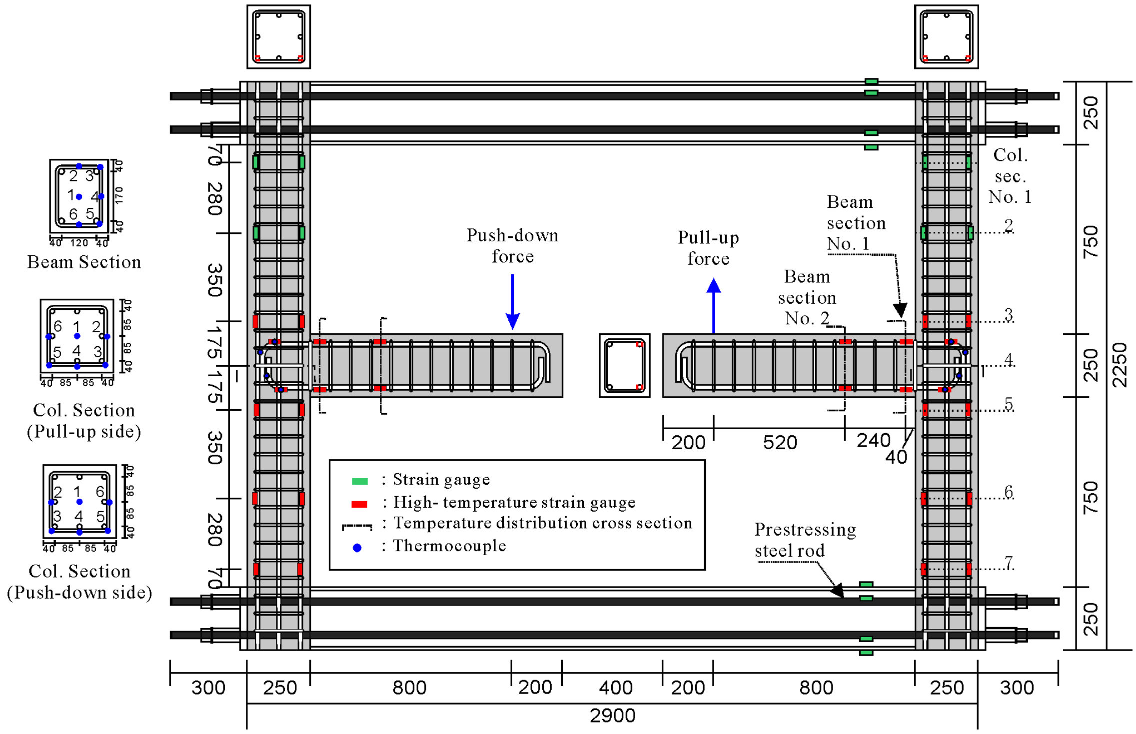


Figure 1: Specimens geometry & arrangement of strain gauges and thermocouples (unit: mm)

Table 1: Concrete mix and its mechanical properties

Age (day)	Cement type	Max size of aggregate (mm)	Slump (cm)	Air content (%)	W/C (%)	Moisture content (%)	Strength (N/mm^2)	Young Modulus (GPa)
28	Normal Portland Cement	20	16.5	4.6	63	5.5	23.9	24.1

Table 2: Reinforcement properties

Type	Location	Area (mm^2)	Yielding strength (N/mm^2)	Tensile strength (N/mm^2)	Young`s modulus (kN/mm^2)
D19 SD345	Beam long. reinforcement	286.5	383	504	192
D16 SD295	Column long. reinforcement	198.6	366	446	190
D10 SD295	Column & beam hoop*	71.3	365	428	189

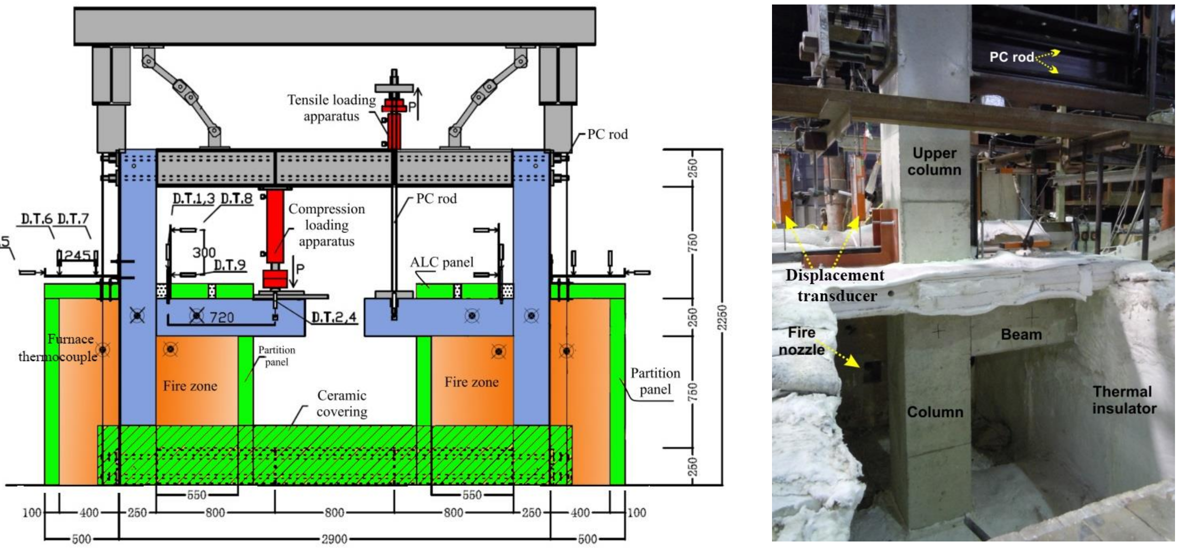
* The hoop interval = 80mm

2.2 Measuring instruments

Figure 1 shows the arrangement of thermocouples and strain gauges. Cross-sections 4 to 7 of each column and 1 and 2 of each beam were instrumented with six thermocouples per a cross-section. Four thermocouples were also installed inside the beam-column joint. However, in this paper the temperature distributions of cross-sections 1 and 2 of beams and 4 of columns are presented. Figure 2a illustrates the arrangement of displacement gauges. In order to measure the deflection of each beam, two displacement transducers (DT), DT1 and DT3, were installed at a section 80mm from the column face (80mm-section) and another two, DT2 and DT4, at 800mm-section, respectively. The relative deflection of each beam was the difference of the recorded values of the 80mm-sections and 800mm-section. To measure the elongation and rotation of each specimen, DT5 to DT9 were installed. Load cells measured the applied load at each beam free end as well.

2.3 Loading and heating procedure

Arrangement of loading apparatuses and furnace are shown in Figure 2a and 2b, respectively. The test was consisted of two phases; in the first phase two equal vertical 17.3kN forces were applied at the left beam end and the right beam end downward and upward, respectively. The loading point in each specimen was at a section which was 800mm from the column face. When the specimens were under the load, the widths of newly formed hairline cracks were measured (less than 0.10mm wide).



(a) (b)

Figure 2: Loading apparatus, furnace, and displacement transducers arrangement (unit: mm)

In the second phase, the heating process started according to the standard fire ISO834 which is shown in Figure 3. Eight thermocouples were measuring the furnace temperatures. The fire zone in each specimen was covering the beam (except for its top face) and the lower column. To keep the load cells out of the furnace, the fire-exposed length of each beam was limited to 550mm from the column face. The lower steel H-beam was thermally insulated with several sheets of insulator and gypsum paste.

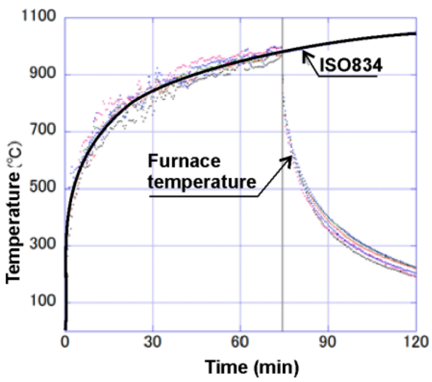


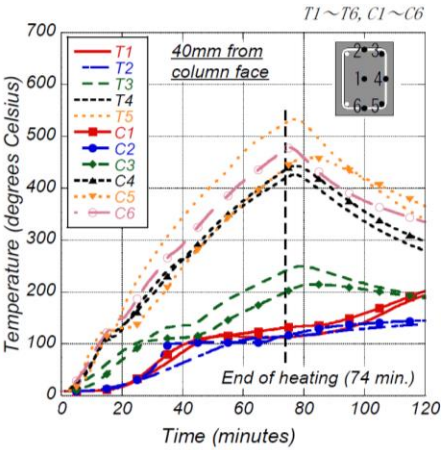
Figure 3: Standard fire ISO834 and furnace temperatures

During the test, through a 10×10cm window on the north side of the furnace, the states of the specimens were being observed. 15 minutes after beginning of the test some cracks appeared at the column base of the pull-up side. Even though no exploding sound was heard, later inspection of the cooled specimens showed the partial explosive spalling of the concrete as the main reason of the large cracks. Due to the large decrease in strength of concrete and yielding strength of reinforcing steels at elevated temperatures large deformations occurred, which could result in sudden collapse. To prevent this and considering the safety matters, based on BS8110 yielding strength degradation model, as soon as the thermocouple No. 5 that was attached to the longitudinal reinforcement at the 80mm-section of the beam recorded the temperature of 560°C (AIJ 2009) the fire was stopped. The test finished in 74 minutes and roughly two hours later the furnace lids were opened for further in-place inspections.

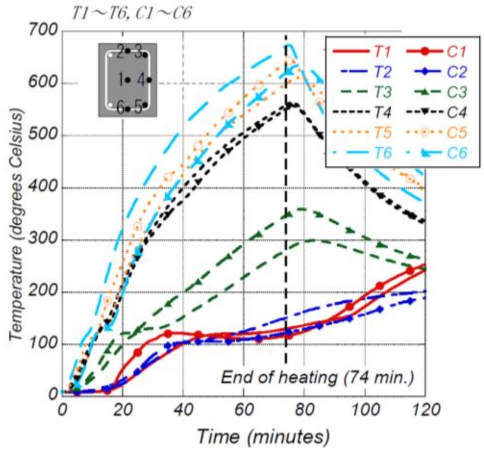
3 TEST RESULTS

3.1 Beam internal temperature distributions

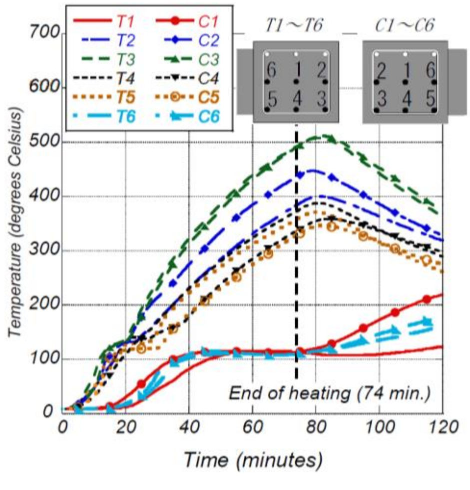
In order to investigate the temperature distributions inside the beam-column sub-assembledges, each of the three sections was instrumented with six thermocouples. T1~T6 belonged to the pull-up side and C1~C6 belonged to the push-down side. The beam cross-sections in each side were located 40mm (40mm-section) and 280mm (280mm-section) from the column face. The column cross-section was aligned to the beam horizontal centroid axis. As can be seen in Figure 4, thermocouple No.1 measured the central temperature of each cross-section, No.2, 4, and 6 measured the hoops temperatures, and No. 3 and 5 recorded the temperatures of the longitudinal reinforcements.



(a) 40mm-section



(b) 280mm-section



(c) Beam-column joint

Figure 4: Internal temperature distribution at different sections

As can be seen in Figure 4a, the bottom longitudinal reinforcement of the beam at the pull-up side reached the temperature 350°C 15min sooner, when the reinforcing steel strength degradation started. This was probably because of the cracks that had formed under the load and widened and extended at elevated temperatures. The effect of crack on fire-exposed reinforced concrete members is under investigation by the authors. At the end of the test the bottom longitudinal reinforcement in the pull-up beam was 90°C hotter than the push-down side. However, the temperature measured at the top longitudinal reinforcement (No.3) in the pull-up side was 40°C lower than that of the push-down side at the end of the tests. Thermocouples No.2 and No.4 (hoops), regardless to the loading direction and the location of cross-section, roughly recorded the same temperatures. However, thermocouple No. 6 of 280mm-section of the pull-up side recorded temperature approximately 100°C higher than that of the push-down side (Fig. 4b). Thermocouple No.6 in the 40mm-section of the pull-up side failed to record any temperature. The 40mm-section absorbed less heat than 280mm-section. This showed a heat flow (heat conduction) from the hotter regions (cross-sections far from the column) to the less hot regions (cross-sections close to the column face).

3.2 Internal temperatures of beam-column joint

Figure 4c shows the internal temperature distributions within the beam-column cross-section. This section was aligned to the centroid axis of the cantilever beam. Thermocouple No.1 was at the centre of the section and No.2 ~ No.6 were attached to the column longitudinal reinforcements. Even though thermocouple No.6 was close to the cracked zone, it measured considerably low temperatures. However, this lower temperature was in compliance with the heat transition from the 40mm-section to the column. Unlike the beams, there were no significant internal thermal differences when the loading direction changed. At the end of the fire test, thermocouples No.4 and 5 and thermocouple No.2 in the push-down side were 40°C and 50°C less hot in the pull-up side, respectively.

3.3 Crack Pattern

After the fire test, an in-place visual inspection for the specimen was carried out. The color of the cooled specimens apparently had turned buff and mild pink. Except for intricate networks of superficial thermally-induced crackings all over the fire-exposed surfaces, several major medium and wide crackings were spotted as shown schematically in Figure 5.

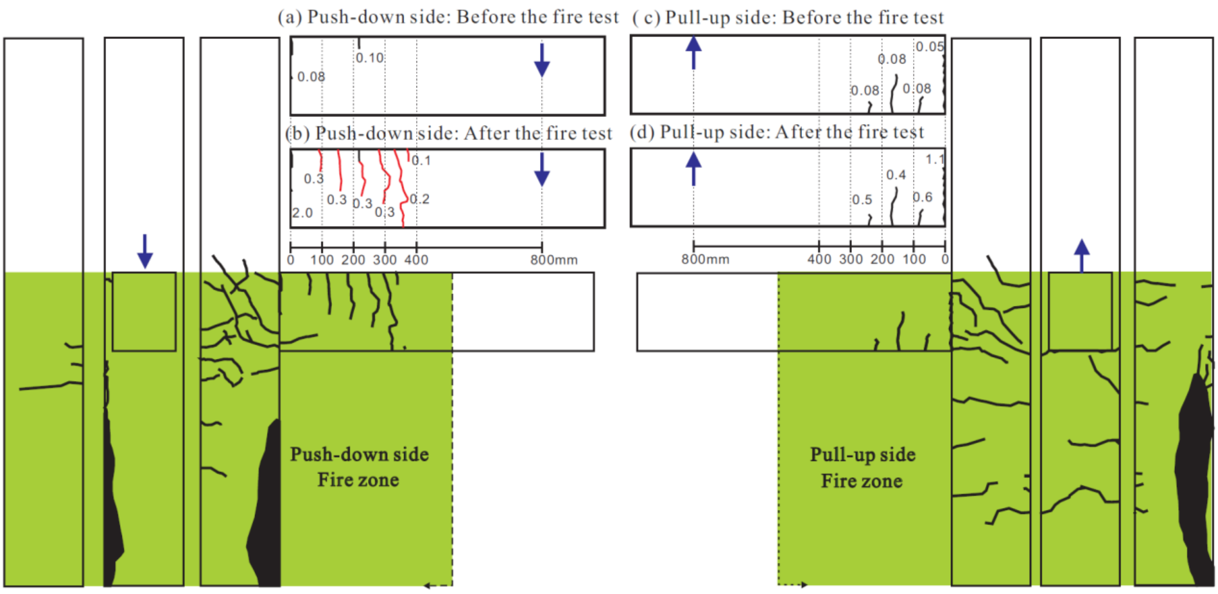
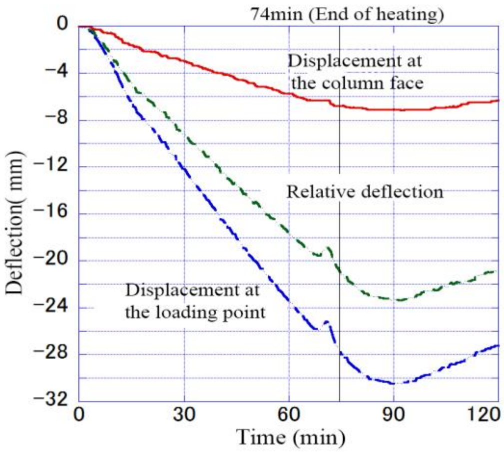


Figure 5: Crack pattern after the fire test (The black zone is the explosive spalling of concrete and the red lines are representing the extention of initial cracks)

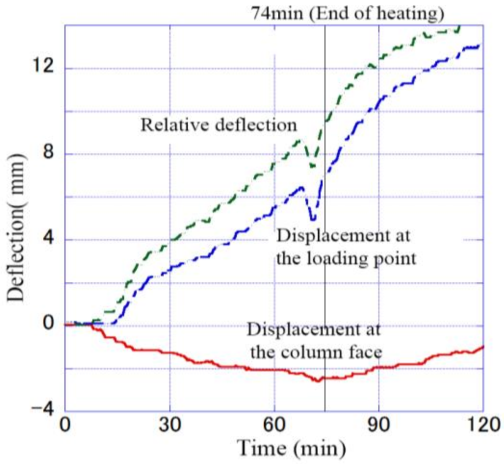
After the initial loading, some hairline flexural cracks appeared in the beams. For example, at the beam-column intersection face in the push-down and the pull-up sides, 0.08mm and 0.05mm wide cracks formed, respectively. These cracks widened and extended in fire condition and other new ones appeared in the beams, beam-column joints, and columns. The maximum crack widths at the end of the test in the push-down and the pull-up sides were 2.0mm and 1.1mm, respectively. In the pull-up side, after 15 minutes, explosive spalling of concrete caused large cracks at the base of the column and as a result, some reinforcing steels of the column were directly exposed to the fire. The explosive spalling of concrete in the column of the push-down side also occurred but the exact time of happening was not detected.

3.4 **Relative vertical deflection**

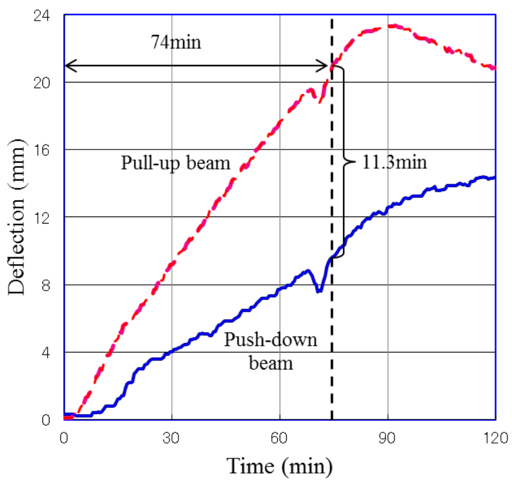
The relative vertical deflection of each beam was the difference of the vertical displacements of two locations on each beam; one roughly at the fixed end (80mm from the column face) and one at the loading point (800mm from the column face). Figure 6a and 6b show the relative vertical deflections of the pull-up and the push-down side beams against time, respectively.



(a) Pull-up side



(b) Push-down side



(c) Comparison

Figure 6: Relative vertical deflection of each beam

Figure 6c is the comparison between the relative deflections of the beams. The positive direction is the downward deflection. As can be seen in the figure, at the end of the test the relative deflection of the pulled up beam was 11.3mm larger than that of the push-down side. That is because of a 326°C thermal difference in the tensile reinforcements (199°C at C3 in push-down side and 525°C at T5 in pull-up side) and caused large degradation in stiffness and strength of the longitudinal reinforcements of the pulled-up beam. 15 minutes after starting the test in the compression side, almost no deflection was recorded. That was because of an upward bending deformation (opposite direction) due to the thermal differences between the top layers (thermally insulated) and the heated bottom layers of the beam. However, in the pull-up side the thermal upward bending and loading led to larger deflections.

3.5 **Elongation and Rotation**

During the fire test, specimens gradually expanded due to the thermal expansion. Figure 7 illustrates the columns' avarage elongations which were recorded by displacement transducers 1, 3, 6, and 7. As can be seen in the figure, the column of the push-down side had smaller vertical elongation at elevated temperatures than the pull-up side due to the subjected downward compression force. In the pull-up side, however, the same directions of vertical elongation and force led to larger vertical expansion in the column.

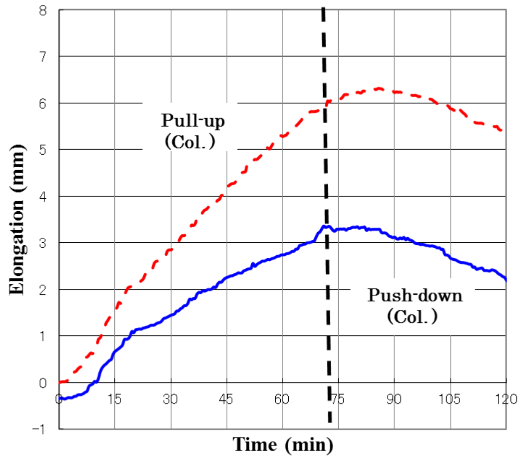


Figure 7: Columns' vertical elongation

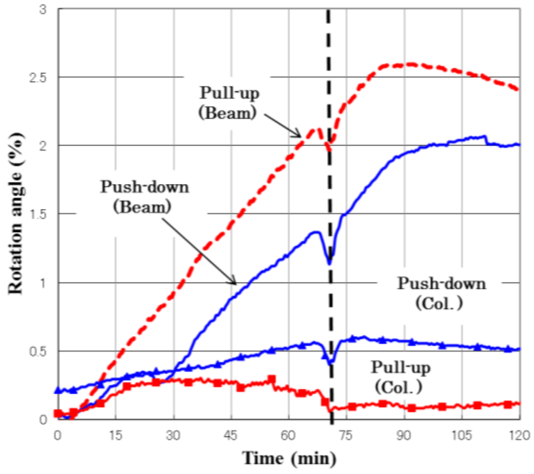


Figure 8: Column and beam rotation

Figure 8 shows the rotation angles of the beams and columns in fire condition, respectively. As for the beams, the explanations discussed in section 3.4 are applicable here as well. The rotation angle of each column was measured in a section belonged to the upper column and close to the beam.

4 CONCLUSION

In this experiment, two external RC beam-column sub-assemblages which were subjected to 17.3 kN service loads were tested in fire condition. The conclusions are as below:

1. The internal temperature distributions within the two beams showed that the pulled-up beam tended to have hotter cross-sections close to the column face. Furthermore, the immediate cross-sections to the columns of both sides tended to keep lower temperatures. It was attributed to the heat transfer occurred from regions of higher temperatures to another region of lower temperature to reach a thermal equilibrium.

2. It is assumed that the cracks that had formed before and during the fire test caused higher internal temperatures at the cracked zones and led to decrease in the strength of both concrete and reinforcing steels. In order to figure out the effect of cracks at elevated temperatures on structural behaviour of RC members, an extensive research by the authors is being carried out.

3. The thermal degradation of tensile reinforcement strength and Young's modulus led to large deflection and this deflection was amplified in the pull-up side by the beam thermal upward bending deformation.

A full 3D finite element analysis of the test has been carried out as well to provide the analytical thermal strain and stress distributions within the specimens. The results of the finite element modelling will be published in a separate literature.

ACKNOWLEDGEMENT

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