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Fire resistance of self-compacting concrete beam-column sub-assemblage

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ABSTRACT: The fire resistance of three full-scale beam-exterior column sub-assemblage specimens, designed according to ACI 318 seismic provisions and subjected to ISO 834 fire test, is presented. Two specimens were made of Self-Compacting Concrete (SCC) and one was made of Normal Concrete (NC). Analysis based on thermal properties of materials suggested in EC2 and ANSYS software was carried out to predict the temperature distribution of specimens. The specimens performed satisfactorily after subjected to three hours ISO834 fire exposure. The significant explosive spalling occurred in about 25 minutes after heating. The beam deflection at the end of heating was approximately 10 times that before heating. The interaction of deformation between beam and column is presented. In the residual strength test, two specimens failed in ductile flexural mode, while another one failed in shear after significant yielding. The predicted and measured temperature distribution of specimens is well correlated.

1 INTRODUCTION

The reinforced concrete structures are inherently fire resistant. However, the spalling of concrete would expose steel reinforcements in a reinforced concrete member during fire hazard, which could further damage the components or structure. The critical factors affecting concrete spalling have been an important subject studied by some researchers (Sullivan, PJE 2002, Hertz 2003, Anderberg 1997, Person 2004). The self-compacting concrete (SCC) is one of the high performance concretes and has been used in some buildings and infrastructures in Taiwan recently. The dense properties and lower permeability of SCC make it susceptible to spalling under severe fire environment. The fire endurance of SCC materials has been one of the major concerns of its use in structures subjected to the elevated temperature (Kodur & Phan, 2007).

The current prescriptive provisions of fire resistant design of buildings are primarily based on the results of single components. The deformation characteristics and associated restraining effect on components are useful data for the validation of computer program used for fire resistant performance based design of structures. The results of half scale reinforced high performance concrete portal frames recently tested by (Xiao et al., 2008) indicated that significant longitudinal elongations of beam and column were observed during the early stage of heating. The existing data for the behavior of sub-assemblage under elevated temperature is still very limited.

The results presented here is part of a series of tests conducted to study the fire performance based design drafts currently developed in Taiwan. The specific objective of this study is to investigate the fire resistant behavior of beam-exterior column sub-assemblage designed according to ACI 318 seismic provisions.

2 EXPERIMENTAL PROGRAM

The exterior beam-column sub-assemblage from a seven-storey reinforced concrete model residential building, designed according to ACI 318 seismic details, was used for the specimens as shown in Figures 1–2. The magnitude and locations of concentrated loads, P1 and P2, applied at the beam of specimens were designed to correlate the moment distribution in beam of the model residential building.

Three full-scale specimens were fabricated vertically as they were constructed in practice. The dimensions of column, beam and spandrel beam are $500 \times 500 \times 2860$ mm, 400 (W) $\times$ 500 (H) $\times$ 7680 (L) mm, and 400 (W) $\times$ 500 (H) $\times$ 600 (L) mm, respectively. The height of column is designed to simulate the point of inflection located at both ends of column as the exterior column deforming in service load condition of the model building.

Two types of concrete, i.e. normal (NC) and self-compacting concrete (SCC), were used. The mix proportion of the concrete materials is shown in Table 1. The cylinder compressive strength was 39 MPa for specimen NC5 and 46 MPa for specimens SCC4 and SCC5 at test date.

The column and beam of specimens were loaded and followed by the ISO 834 standard temperature test for three hours in a beam-column composite furnace built in Tainan, Taiwan. The dimensions of the furnace are 4 m wide by 8 m long. The depth of furnace for column is 5 m, while that for beam is 4 m. There are totally 30 burners, using natural gas and air as fuel, on

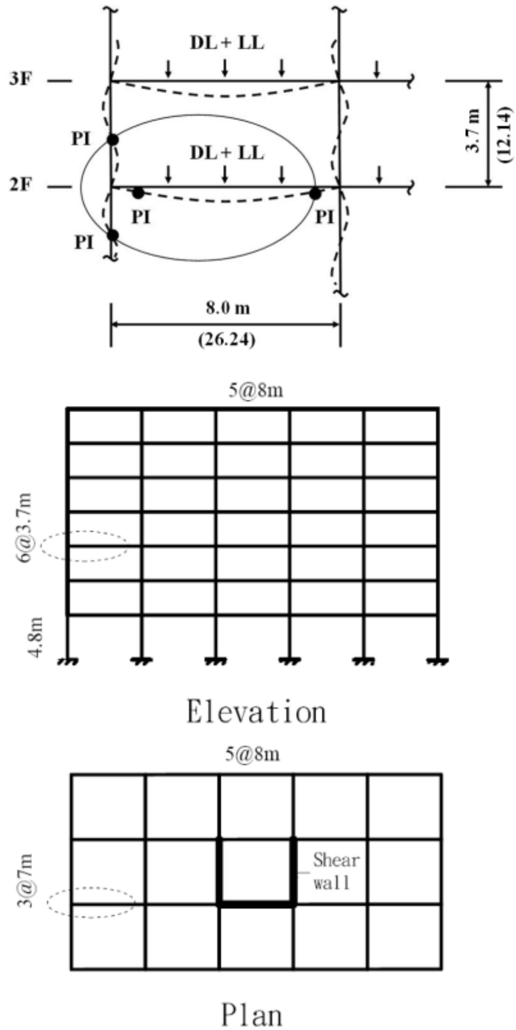


Figure 1. Model of test specimen and residential building.

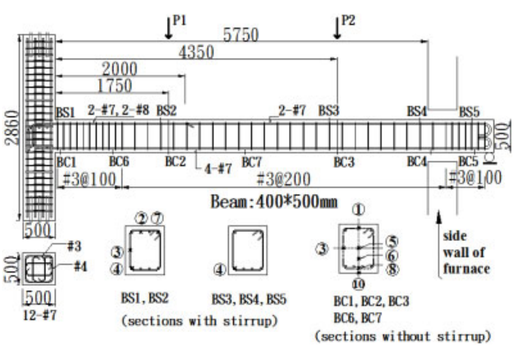


Figure 2. Reinforcement details and locations of thermal couples.

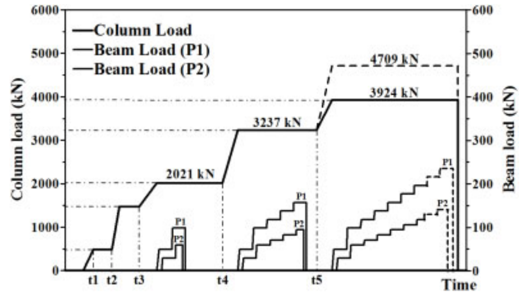


Figure 3. Load procedure.

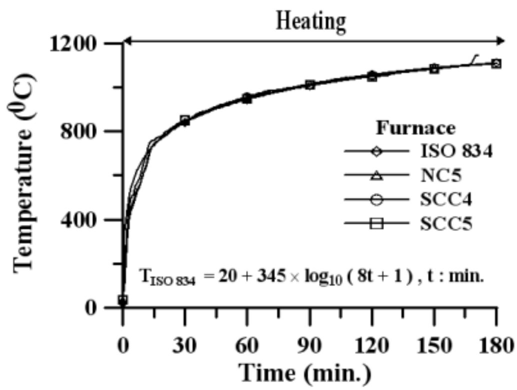


Figure 4. Average furnace temperature in specimens.

Table 1. Mix proportions of specimen materials (kg/m^3)

Ratio	NC	SCC
Water	182	166
Cement	190	200
Slag powder	114	160
Fly ash	76	110
w/binder	0.49	0.35
Fine aggregate	823	830
Coarse aggregate	938	790
Superplasticizer	3.8	5

both sides of the long wall of furnace for generating the computer controlled temperature. The recorded average furnace temperatures of the three specimens were very close to ISO834 standard curve except in the early 15 minutes of heating as shown in Figure 3. The top of beam was wrapped with 50 mm thick ceramic fiber to simulate the 150 mm thick concrete slab. The exterior half of the column faces was also wrapped with 50 mm ceramic fiber to simulate the condition of three-face exposure to fire. At the end of heating, the loads on column and beam were retained and the data acquisition was turned to automatic mode to continue monitoring the recovery of deformations of specimens during the cooling stage. Fifteen hours after cooling the specimen was unloaded and the door of furnace was opened. One week later, the residual strength test was conducted. The loads applied to column and beam was increased stepwise in magnitude as shown in Figure 4, for the observation of overload behavior of specimens and avoiding the premature failure in column before beam.

3 ANALYSIS

A 3D finite element model is developed using the software ANSYS and the thermal properties of materials suggested in EC2 to predict the behavior of test specimen under combined actions of applied loads and elevated temperature effect. In this study only the results of temperature distribution inside the specimen is presented.

4 RESULTS AND DISCUSSIONS

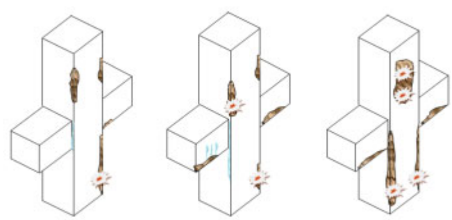
4.1 *Concrete spalling*

During approximately the first twenty five minutes of heating, concrete spalling were observed along the bottom corners of beam as shown in Figures 5a to c. The debris of spalled concrete was also found spreading at the floor of beam furnace. Severe spalling was found at the bottom of beam near column in all specimens. In this study, normal concrete specimen NC5 had more spalling at bottom of beam than SCC specimens SCC4 and SCC5. The corner spalling of lower column in the three specimens, shown in Figures 5d to f, were observed approximately in the first 18 minutes of heating. Following that spalling also occurred at some corners of beam-column joint, and at surface and corners of upper column.

(a)NC5

(b)SCC4

(c) SCC5



(d)NC5 (e) SCC5 (f) SCC4

Figure 5. Spalling of concrete.

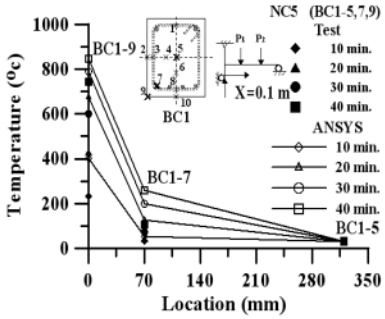
4.2 *Temperature distributions inside specimens*

Most of concrete spalling occurred in the early twenty five minutes of heating. The temperature gradients in beam sections near joint face and load points at early stage of heating are shown in Figures 6a-c, respectively. The temperature of longitudinal reinforcement in lower column close to joint was 580° at 180 minutes as shown in Figure 6d. The severe spalling did not expose the corner longitudinal reinforcement. The maximum temperature of bottom corner longitudinal reinforcement and vertical leg of stirrup in specimen SCC5 was 583° and 530°, respectively, at the end of heating as shown in Figure 7.

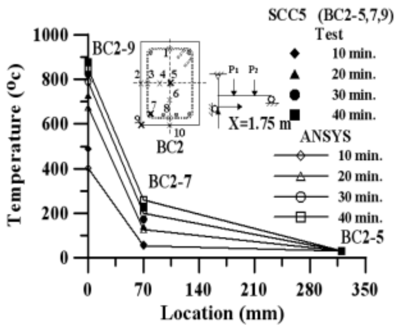
The maximum recorded temperature of longitudinal tension reinforcements mentioned above is slightly over the 550° limit at single point specified in Taiwanese Standard CNS 12514.

4.3 *Deformations during heating stage*

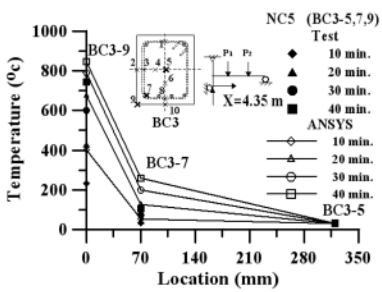
The deformed shape constructed by vertical displacements recorded at two load points of beam before and during heating are shown in Figure 8. More beam deformations occurred in the early thirty minutes of heating, and the maximum value was approximately 60 mm at the end of 180 minutes of heating, which was ten times that at room temperature. The increase of beam deformations was much smaller from thirty to sixty minutes of heating, which is believed due to the lower temperature rise caused by phase change of moisture inside concrete. Similar results were also recorded in other two specimens. The beam elongated toward both ends during the early eighty minutes, as shown in Figure 9, and then most of the elongations



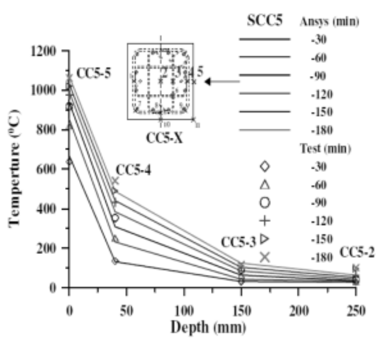
(a) Near joint (Specimen NC5)



(b)At load point P1 of beam (Specimen SCC5)



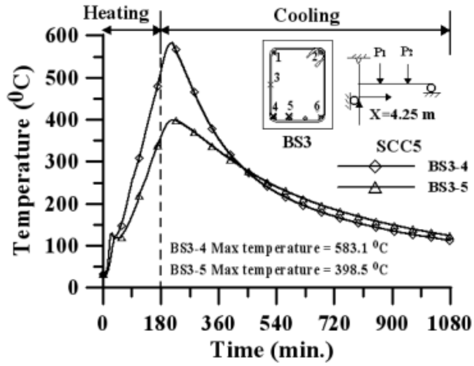
(c)At load point P2 of beam (Specimen NC5)



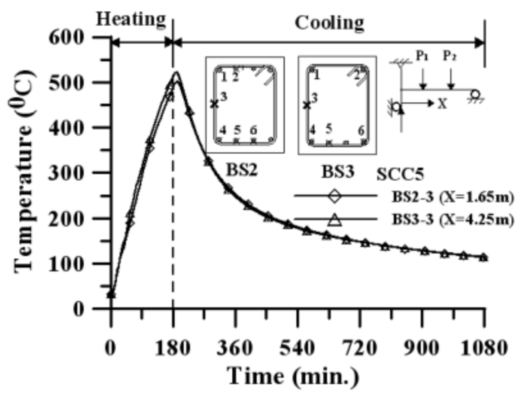
(d) At top of lower column (Specimen SCC5)

Figure 6. Temperature gradients of concrete.

occurred at far end of beam due to the inward deformations of column subjected to three-face heating. The total elongation of beam in specimen SCC4 was 34 mm after 170 minutes of heating.



(a)Temperature of longitudinal reinforcement



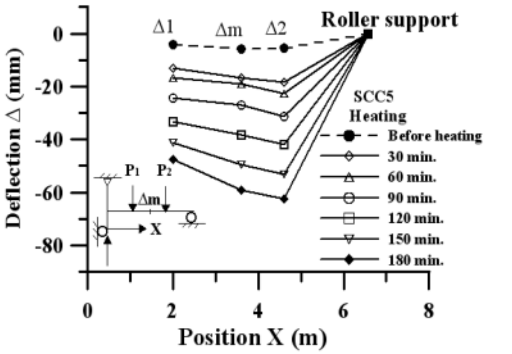
(b)Temperature of vertical stirrup

Figure 7. Temperature variations of reinforcement.

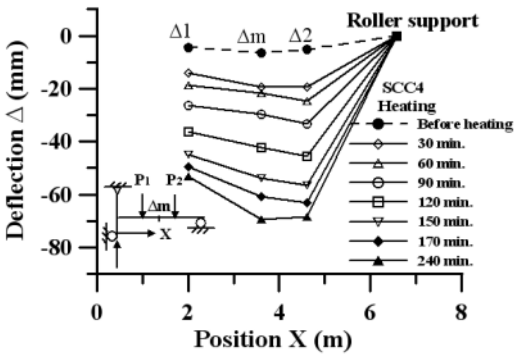
The rate of vertical displacement at load point was further calculated for every five-minute interval, and then the maximum value was picked within every thirty-minute interval as shown in Figure 10. The maximum value was 0.72 mm/min. in the first thirty minute at load point P2 of the specimens, which is far below the limit $L^2/90000d$ specified in Taiwanese Standard CNS 12514, i.e. 11.3 mm/min. in this study.

4.4 *Behavior during residual strength test*

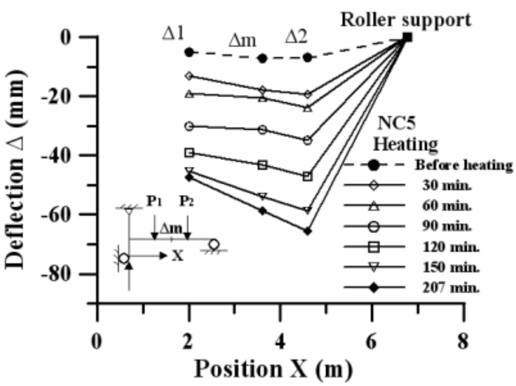
The flexure-shear cracks at top surface of beam near joint and slight shear cracks at load point P1 were observed after heating test. The shear cracks were also observed in the beam tests reported by (Ellingwood & Lin, (1991)). As mentioned previously in Experimental program, the loads applied to beam and column were increased stepwise in residual strength test to monitor the overload behavior of specimens. Figure 11 shows the variations of displacement at load point P2 of beam with respect to various total beam loads. The results were very close in three specimens, which show that specimens made of normal concrete and self-compacting concrete behaved quite closely after three hours fire exposure.



(a) Specimen SCC5



(b) Specimen SCC4



(c) Specimen NC5

Figure 8. Deformed shape of beam during heating.

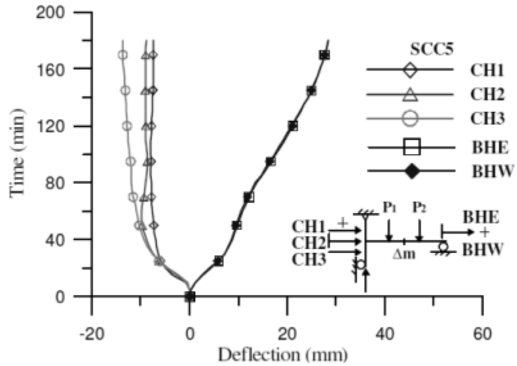


Figure 9. Elongation of beam and horizontal displacement of column (Specimen SCC5).

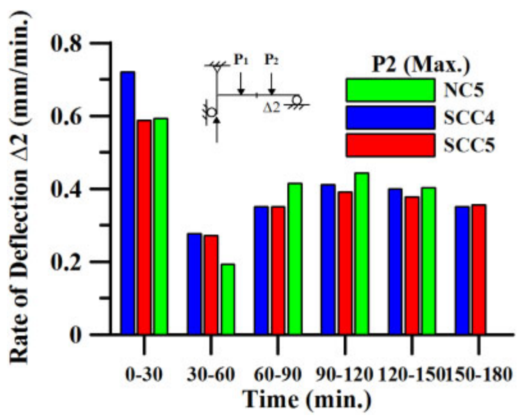


Figure 10. Rate of deflection at load point P2.

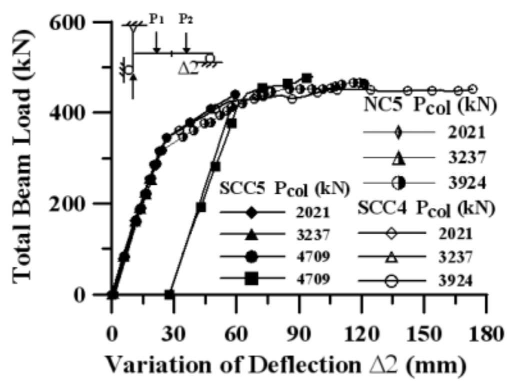


Figure 11. Variation of vertical displacement at load point P2 during residual strength test.

The ductile flexural failure with concrete crushing at top of beam near load point P2 occurred in specimens SCC4 and NC5. However, the specimen SCC5 failed in diagonal tension failure within the region of load point P1 and joint face after exhibiting significant yield behavior. The location of shear failure occurred almost right at the start of stirrup spacing changed from 100 mm to 200 mm. The reason of causing the unfavorable shear failure appears to be due to higher column load applied in specimen SCC5. Further studies on the residual flexural and shear strength of fire damaged beam are still working.

The upper and lower columns exhibited slightly nonlinear moment-curvature relationship at the overload stage as shown in Figure 12.

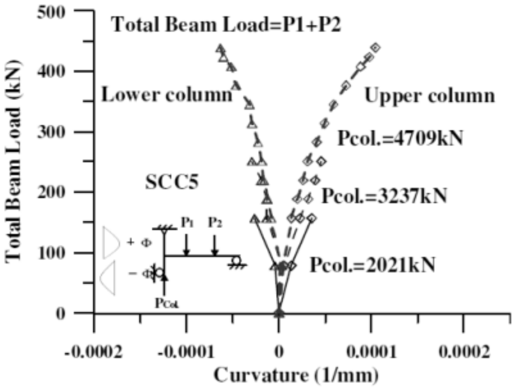


Figure 12. Variation of moment-curvature relationship of upper and lower columns during residual strength test.

5 SUMMARY AND CONCLUSIONS

Three full-size beam-column sub-assemblage specimens designed according to ACI 318 seismic provisions were subjected to combined actions of applied loads and ISO 834 elevated temperature for three hours. The following conclusions are drawn based on the test results.

(1) The specimens behaved satisfactorily under ISO834 standard fire exposure for three hours.
(2) Most of the concrete spalling occurred along the bottom edge of beams and corner of lower column during the early twenty five minutes of heating. Relatively more spalling was observed at bottom of beam in normal concrete specimen NC5.
(3) The concrete spalling did not expose the longitudinal reinforcements in beam and column. However, the maximum temperatures recorded at several locations in longitudinal reinforcements of beam and column reached 580°, which exceeds the limit 550° at single point specified in Taiwanese Standard CNS 12514.
(4) Most of the vertical displacements in beam occurred in the first thirty minutes of heating, and then the increase of vertical displacement decreased during the thirty to sixty minutes due to the low temperature rise in beam.
(5) The normal and self-compacting concrete specimens behaved quite closely in their load- displacement relationships at load point in residual strength test.
(6) Two specimens failed in ductile flexural mode and one specimen failed in unfavorable diagonal shear after exhibiting significant yield behavior in beams. Further studies are still needed for the residual flexural and shear strength of members.

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