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Experimental study of reinforced concrete columns damaged by fire following an earthquake

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ABSTRACT

This study investigates the cyclic behavior of reinforced concrete columns that have been exposed to a major fire following a minor to moderate earthquake. Eight specimens with varying column bar diameters and varying confinement were tested under two primary scenarios: with and without fire damage after the earthquake. A typical column on the lower floor of a building was selected as the prototype column. The nonlinear response of the column was simulated numerically by subjecting it to a moderate earthquake. The computed top displacement of the column was recorded and served as the initial loading on the column specimens to represent earthquake-induced damage. Four of the eight specimens were subjected to standard cyclic loading to assess the capacity of undamaged columns. The remaining four columns were assumed to sustain a major fire and the fire loading was simulated by heating the columns using a stackable electrical furnace. Temperature in the furnace followed the ISO 834 standard fire curve and was controlled using a combination of electricity and gas. Columns that were subject to earthquake-induced fire experienced significant cover spalling. Compared to columns that did not experience fire, the fire-damaged columns experienced a loss of strength and stiffness. It was also observed that columns with smaller diameter longitudinal reinforcement and better confined columns exhibited better seismic performance. In this paper, only results from two typical column tests are presented.

1. INTRODUCTION

In current design codes, earthquakes and fires are treated as independent events and earthquake-induced fire is not taken into account. It is important to assess the residual capacity of load-bearing elements that have been subjected to high temperature loading following an earthquake-induced fire. Fire following an earthquake

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is quite common because of the presence of utility lines in urban communities. For example, fires following the 1994 Northridge and 1995 Kobe earthquakes resulted in significant damage.

Several experimental and theoretical studies investigating the performance of RC columns under fire conditions or after fire exposure can be found in the literature. Experimental research indicated that strength, stiffness and load-bearing capacity decreased after high temperature (Chen et al. 2009). Lie (1989) and Dotreppe et al. (1997) both carried out a large-scale experiment to verify fire resistance of RC columns. The experiments studied the influence of parameters such as load level, area and shape of cross section, amount of steel reinforcement, etc. on the behavior of fire resistance of RC columns. Wu et al. (2007) conducted cyclic tests of RC columns after exposing the columns to high temperature and found that deformability and energy-absorbing capacity decreased gradually with increasing temperature. Mostafaei et al. (2009) studied the residual strength and seismic load capacity of RC columns after fire exposure and developed a nonlinear finite element analysis program to estimate the post-fire axial and lateral performance of RC columns.

Although current research studies have examined damage to RC columns subjected to earthquake and fire loading independently, earthquake-induced fire resistance of structures remains an open problem. This paper provides valuable test data pertaining to the seismic behavior of RC columns following earthquake-induced fire. The main objectives of this paper are: a) to study the reserve seismic capacity of large-scale RC columns after earthquake-induced fire; b) to clarify the effects of hoop spacing and diameter of longitudinal steel bars on seismic performance of RC columns subjected to fire following an earthquake.

EXPERIMENTAL PROGRAM

Eight full scale columns were tested to investigate the cyclic behavior of RC columns following post-earthquake fire. Of these, four columns served as benchmark specimens and were not subjected to an earthquake-fire sequence. All specimens were subjected to a constant axial stress of $n=N_o/N_u = 0.3$, where N_o is the applied axial load and N_u is the axial compressive capacity of the column. The effect of two primary parameters were investigated: diameter of longitudinal bars and hoop spacing. In this paper, only the results of one set of columns are presented.

All columns were 350×350 mm square in cross section with a 25 mm concrete cover and the height was 1.5 m from the point of lateral load to the top of the foundation. The column specimen whose results are presented here had a reinforcement ratio of 2% with 8 - 20 mm diameter bars. The transverse reinforcement consisted of 10 mm diameter bars with a spacing of 60 mm. The tie spacing satisfied the requirements of ACI 318-11 (2011). Each set of transverse reinforcement consisted of a peripheral hoop with 135-degree hooks and a pair of cross ties with a 135-degree hook at one end and a 90-degree hook at the other end. The 90-degree hooks were alternated for the cross ties throughout the height of the column. All the specimens were designed and constructed with a stiff foundation of 2×0.7×0.42 m with adequate reinforcement to eliminate any premature failure during testing.

The average yield strength f_y of longitudinal reinforcement was 442 MPa and that of the transverse reinforcement was 433 MPa. The average concrete compressive strength was 44.0 MPa based on compression tests on concrete cubes (150 mm) at the time of testing. The elastic modulus of concrete was 3.3×10^5 MPa based on concrete cylinders 150 mm in diameter and 300 mm in height.

2.1 Test setup

The experiment was carried out in the Key Laboratory of Building Safety and Energy Efficiency at Hunan University. The foundation of specimen was mounted vertically using two stiff steel reaction beams. The testing system utilizes two hydraulic jacks with 1500 kN capacity to provide a constant axial load. An actuator with a capacity of 1000 kN and a maximum displacement of 600 mm for cyclic loading is mounted horizontally to the reaction frame. Load cells and displacement transducers were mounted on the actuators to measure the applied lateral force and lateral displacement at the point of loading. During testing, the applied axial load becomes inclined and this inclination causes a horizontal component of applied axial load which is significant compared. For this reason, the horizontal component is subtracted from the horizontal actuator load to obtain the true lateral force applied to the column.

2.2 Loading procedure

Specimen 1 was directly subjected to cyclic loading without pre-damage due to earthquake and fire while Specimen 2 experienced moderate damage due an imposed seismic load history followed by high-temperature loading corresponding to a standard fire. Specimen 2 was then subjected to the same cyclic load history as Specimen 1. The cyclic loading was controlled by lateral displacement. Initially, three single cycles corresponding to an increment of 0.25% drift ratio were applied. And then three cycles were imposed at drift ratios of 1%, 1.5%, 2%, 3%, 4% and 6%.

To establish the displacement load history for Specimen 2 due to a moderate earthquake, the acceleration time history from the Whittier earthquake (Record 602) was selected from the PEER database and applied to a numerical model of the column. The numerical simulation was carried out using the open-source structural analysis software platform OpenSees (2017). The record was scaled such that the spectral acceleration at the fundamental period of the system was equal to the spectral value of the design spectrum of the Chinese code GB50011 (2010). Following the application of the seismic load, Specimen 2 was heated for one hour in a stackable electric furnace to simulate the fire induced by the earthquake. The furnace heating followed the ISO-834 standard fire curve, and the ambient temperature at the start of heating was 20℃. After cooling down to room temperature, the cyclic loading protocol was applied to the column. The axial load was maintained constant during the earthquake, fire, cooling and cyclic loading. Cyclic loading was stopped when lateral load deteriorated to less than 85% of the maximum lateral load.

TEST OBSERVATIONS

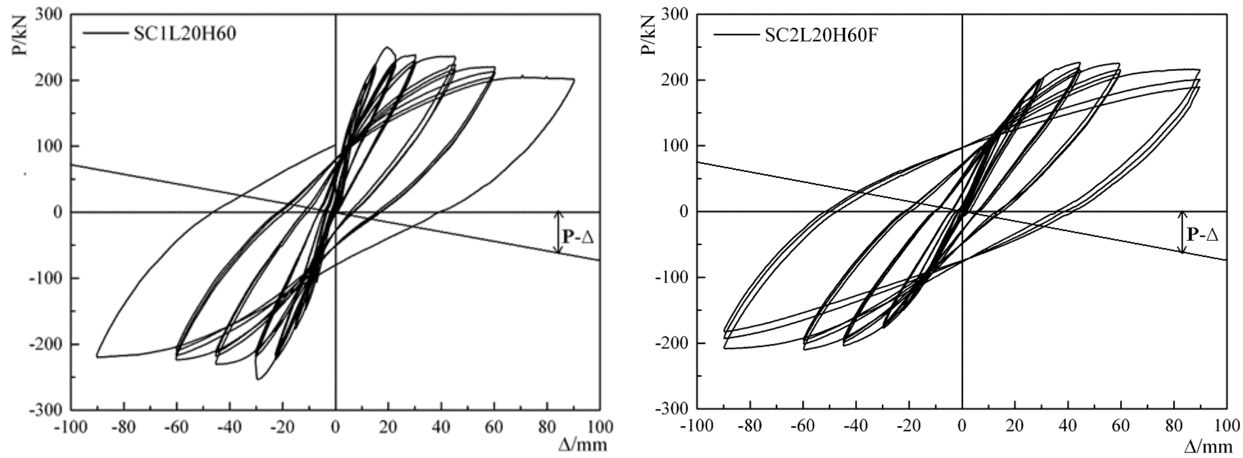
Flexural cracks occurred in Specimen 2 following the application of the simulated earthquake loading (Fig. 1a). Vertical cracking also occurred on the surfaces at the four corners and the concrete cover at the 4 corners of the column base peeled off slightly. The columns was then heated for one hour in the furnace after earthquake loading. During the fire test, water vapor from the specimen escaped out when the furnace temperature reached 500℃. As the furnace temperature increased, a strong steam flow was observed when the temperature went up to 800℃. Finally the water vapor vanished gradually when the furnace temperature reached 900℃. After the temperature exceeded 600℃, continuous crackling was heard, including several big intermittent popping sounds. Explosive spalling of the concrete cover also occurred exposing the longitudinal and transverse reinforcement (Fig. 1b).

(a)

(b)

Fig 1. Damage state following earthquake and fire: (a) flexural cracking after seismic loading; (b) spalling damage after fire loading

Initially, minor horizontal flexural cracks formed at the lower part of specimen 1 at a drift ratio less than 0.5% which extended towards the neutral axis when the drift ratio increased to 1%. The peal lateral force was attained at a drift of 2.0%. With increasing displacement, concrete spalling and concrete crushing followed and the strength gradually degraded. Crushing of the confined core occurred at a drift of 6.0% followed by buckling of the longitudinal bars. The load-deformation response of the specimen is shown in Fig. 2a. In the case of specimen 2, since there was extensive spalling due to the previously imposed seismic and fire loading, cracks were not visible till the drift exceeded 2.0%. The peak lateral force was attained at a drift of 4.0%. When the drift ratio increased to 6%, the lateral force declined rapidly and the crushing of the core was more dramatic. The final failure mode was similar to specimen 1 with longitudinal bar buckling following damage to the core concrete. The cyclic response is shown in Fig. 2b.



(a)
(b)

Fig. 2. Cyclic response of columns: (a) Specimen 1; (b) Specimen 2

CONCLUSIONS

Results of testing on 2 column specimens were presented in the paper. One column was subjected to standard cyclic testing with increasing drift while the second specimen was pre-damaged due to a moderate earthquake followed by fire prior to application of the cyclic load. The following conclusions can be drawn from observations during the testing.

1. The failure mode of both specimens was similar – final failure was caused by buckling of longitudinal steel bars and crushing of confined concrete core.
2. Severe spalling of the concrete cover occurred in the specimen during fire loading.
3. Degradation of strength and ductility was more severe in the case of the specimen pre-damaged by earthquake and fire loading.
4. The cumulative energy dissipation of the fire-damaged columns was slightly higher – this may be due to the fact that air gaps in the specimen after moisture in the columns was removed during fire loading contributed to more frictional resistance.

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