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Experimental studies on load redistribution behavior of multi-storey by multi-bay substructures

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ABSTRACT

Progressive collapse behavior and load redistribution path of reinforced concrete (RC) frames subjected to sudden or general missing column scenarios had been investigated extensively. However, it should be emphasized that majority of existing tests are focused only on the single-storey beam-column sub-assemblage or substructures. It should be noted that the buildings are commonly multi-storey. The reliability of single-storey substructures to represent the true multi-storey building is questionable. In single-storey substructures, the load redistribution is always horizontal. However, in multi-storey buildings, the load could be redistributed horizontally and also vertically. Thus, for investigation of the load redistribution behavior of conventional multi-storey buildings more realistic, a series of multi-story and multi-bay substructures were tested in this study. The sequence of plastic hinges and fracture of rebar reinforcements is captured. The different behavior of true multi-storey building and single storey substructure subjected to the loss of an exterior column is quantified.

1. INTRODUCTION

Progressive collapse is defined by the ASCE/SEI 7 (ASCE/SEI 2010) as “the spread of an initial local failure from element to element, which eventually results in the collapse of an entire structure or a disproportionately large part of it”. The catastrophic consequences of historic events demonstrated that the tragedy of progressive collapse includes both properties and life loss. However, designing structures to be very strong by considering of the scenario of one or couple columns missing in normal design is unrealistic and uneconomic as the progressive collapse events are of low probability. As such, exploring the inherent potential of the structures (i.e the secondary load resisting mechanisms) to mitigate progressive collapse is an alternative way to mitigate the collapse risk of the structures. The secondary load carrying mechanisms include tensile catenary action (TCA) and compressive arch action (CAA) developed in beams as well as tensile membrane action (TMA) and compressive membrane action (CMA) developed in slabs.

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In the past decade, a number of studies had been carried out to evaluate the effectiveness of the secondary load resisting mechanisms for progressive collapse mitigation. Several researchers (Orton et al. 2009, Sadek et al. 2011, Choi and Kim 2011, Qian and Li 2013, and Qian et al. 2015) have studied CAA and TCA in resisting progressive collapse. They confirmed that for beam-column subassemblages subjected to the loss of an interior column scenario, considerable CAA and TCA could develop in the beams to resist progressive collapse (Sadek et al. 2011, Qian et al. 2015). If the missing column is penultimate, which is just nearby the corner column, the effectiveness of CAA and TCA may reduce as the corner column may not be able to provide enough axial constraints to the beams (Qian and Li 2011 and Qian and Li 2012a). If the missing column is a corner column, then the resistance from CAA and TCA is limited and it was pointed out that the existing tie-strength method proposed by DoD (2009) should be revised for the scenario of the loss of a corner column (Qian and Li 2013). Recently, the efficiency of TMA and CMA developed in RC slabs for progressive collapse prevention was also evaluated experimentally by several researchers (Qian and Li 2012b, Li et al. 2014, Qian et al. 2015, Qian and Li 2016 and Ren et al. 2016). They concluded that considerable TMA could develop in the slabs to resist collapse no matter the lost column is located at interior, exterior or corner of the building. However, the contribution of CMA is dependent on the location of missing columns. If interior or exterior column is lost, CMA is significant and cannot be ignored in progressive collapse design (Qian et al. 2015, Ren et al. 2016). However, when the missing column is located at the corner of the frame, CMA is not so effective (Qian and Li 2012b).

It should be noted that above conclusions are derived from tests with simplified single-storey substructures or beam-column sub-assemblages. In reality, the frames with multi-storey may have different efficiencies for the secondary load resisting mechanisms to redistribute the load, which is initially resisted by the lost columns. Thus, it is necessary to carried out tests with multi-stories beams to quantify the behavior of load redistribution in real multi-storey frames.

2. EXPERIMENTAL PROGRAM

2.1 Specimen Design

Two one-quarter scaled multi-bays by multistory RC frames are designed and fabricated in this study: NS and SD. A two-dimensional (2D) interior frame is extracted from the building and investigated experimentally after the loss of a column. The behavior of both exterior and interior joints influenced by the compressive arch action (CAA), tensile catenary action (TCA) could be investigated as the lost column is located just beyond the exterior column. The prototype building is an eight story office building with story height of 3600 mm in the first story and 3300 mm in upper stories. The spans of two orthogonal directions are 7200 mm. The design live load is 2.0 kN/m^2. The dead load including self-weight is 6.4 kN/m^2. For Seismic Designed Specimen SD, it is assumed that the prototype building is located on a D class site where the short period and one-second period spectral accelerations SD_S and SD_1, are 0.48g and 0.35g, respectively. Fig. 1 illustrates the typical reinforcement layout of Specimen SD. As shown in the figure, the transverse reinforcements are hoop stirrups with 135-degree bends. Two transverse reinforcements are placed in the joint region to resist shear force at the joint. In addition, the beam longitudinal reinforcements are doubly continuous without considering the curtailment of reinforcement. As the specimens with three-story by two-bay are constructed, the allowable scale is one-quarter in consideration of the capacities of laboratory facilities. As shown in Fig. 1, the span of the specimen is 1800 mm. The story height of the specimen is 900 mm in the first storey and 825 mm in upper stories. To fix the frame, the foundations are enlarged at the column bases. The cross sections of the beam and column are 90 mm × 140 mm and 150 mm × 150 mm, respectively. The concrete clear cover is 7 mm and 10 mm in beam and column sections, respectively. The side columns are fabricated with whole three stories while the middle one is only fabricated the upper two stories to simulate the ground center column is removed due to extreme loading before testing. For Nonseismic Designed Specimen NS, identical span, story height, beam, and column dimensions are selected for easier comparison. However, the reinforcement details are quite different from the seismic designed one. The hoop stirrups with 90 degrees bends are utilized for transverse reinforcements. No transverse reinforcements are placed in the joint region. Moreover, curtailment of reinforcement is considered well based on the provision in ACI 318-08 (2008).

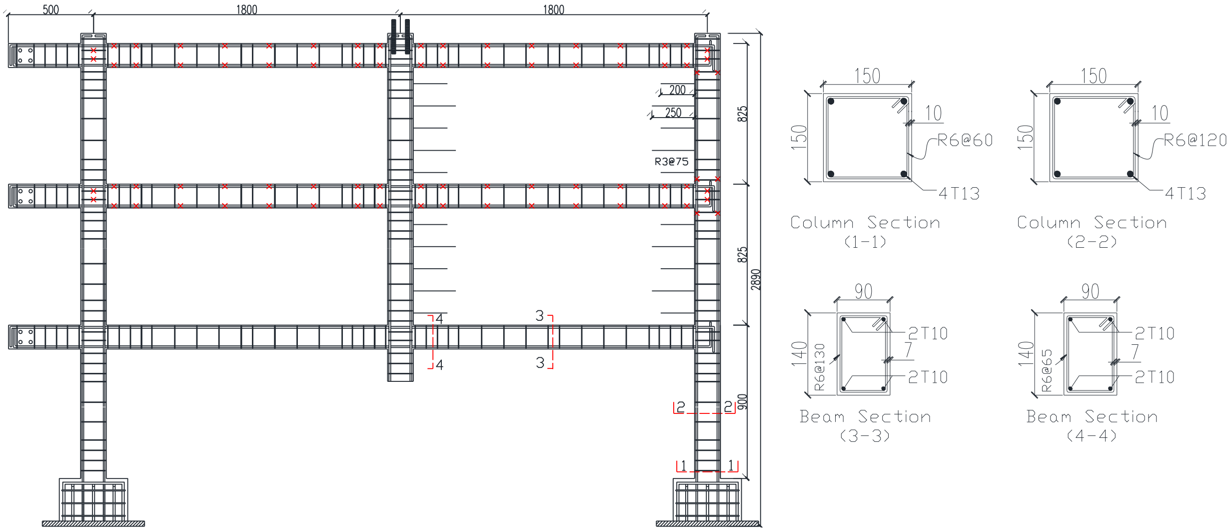

Fig. 1 Dimensions and reinforcement details of Specimen SD

2.2. Material Properties

The targeted compressive strength of the concrete is 30 MPa at age of 28 days. Average cylinder compressive strengths measured on the days of testing for Specimens NS and SD are 32.1 MPa and 33.9 MPa, respectively. The yield strength of R3, R6, T10, and T13 are 417, 449, 515, and 534 MPa, respectively. The ultimate tensile strength of R3, R6, T10, and T13 are 479, 537, 594, and 618 MPa.

2.3. Setup and Instrumentations

Fig. 2 illustrates the typical experimental setup of the tests. As seen in Fig. 2, the specimens are fixed to the strong floor of the lab. through the foundations, which is constructed monolithically with the side columns. A hydraulic jack with 600 mm stroke is utilized to apply vertical load at the top of the center column. A load cell, which is installed above the hydraulic jack, is used to measure the applied vertical load. A steel column together with a special designed steel assembly is installed to prevent undesired out-of-plane movement of the specimen. Three rollers are installed horizontally at the extension part of the frame to simulate the horizontal constraints of the beams in the surrounding bay.

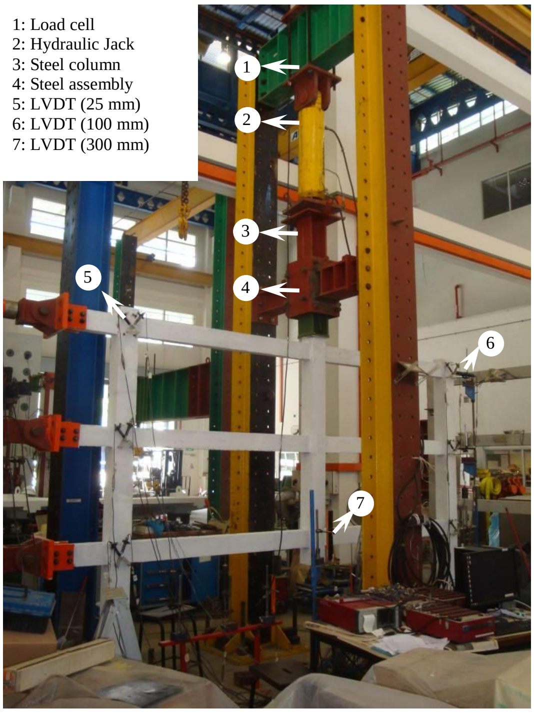

Fig. 2 Specimen SD ready for test

3. EXPERIMENTAL RESULTS

General Behavior

Specimen NS – When applied the vertical load, the first crack is formed at the beam ends near to the center column (BENC). Then, the cracks are also formed in the cut-off points of top beam longitudinal reinforcements. In general, the cracks occurred in the first floor earlier than those in the third one. Shear cracks are occurred in the exterior joint in the first floor when the vertical displacement reached 280 mm while there is no crack in the interior joints. The load-displacement curve of Specimen NS is illustrated in Fig. 3. As shown in the figure, the yield load (YL) of the specimen is measured to 28.4 kN at the displacement of 21 mm while the first peak load (FPL) of 31.9 kN is measured at the displacement of 38 mm. Thus, the FPL is larger than YL only by 12 %. This indicates the contribution of compressive arch action (CAA) in this specimen may not be so effective. When the displacement reached 153 mm, the load resisting capacity of the specimen begins to re-ascend. However, the re-ascending of the load is quite mild. When the displacement reached 286 mm, the specimen achieved the ultimate capacity (UL) of the specimen in large deformation stage. However, the UL is measured only 28.9 kN, which is about 91 % of its FPL. The failure mode of Specimen NS is shown in Fig. 4.

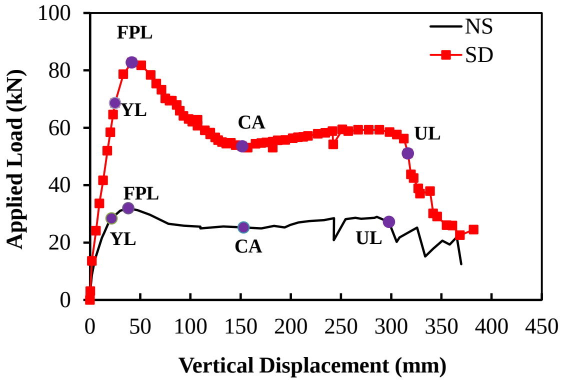

Fig. 3 Comparison of the load-displacement curves

Fig. 4 Failure mode of Specimen NS

Fig. 5 Failure mode of Specimen SD

Specimen SD –Comparing to Specimen NS, the cracks occurred in the beams of Specimen SD are more even and the crack width in this specimen is much thinner at the same vertical displacement. Similar to Specimen NS, the crack pattern of the beams in each storey is almost identical. Fig. 3 gives the load-displacement curve of Specimen SD. As shown in the figure, The YL of 68.6 kN, which is 242 % of that of Specimen NS, is obtained at the displacement of 25 mm. When the displacement reached 42 mm, the FPL of 82.7 kN, which is 259 % of that of Specimen NS, is measured. When the displacement reached 152 mm, the load resistance stayed almost constant with further displacement. The UL of 59.3 kN is achieved at the vertical displacement of 317 mm. The failure mode of this specimen is illustrated in Fig. 5.

4. CONCLUSIONS

Curtailment of tension reinforcement in beam members will change the failure mode of the bare frame but it will not prevent the development of TCA. The insufficient horizontal constraints and possible shear failure at the exterior joints detriments the efficiency of the CAA and TCA developed in the multi-storey bare frames significantly. The initial stiffness, yield load capacity, first peak load capacity, and ultimate load capacity in Specimen SD is about 158 %, 242 %, 259 %, and 205% of that in Specimen NS. The horizontal movement behavior of the exterior joints in different stories indicated that the horizontal constraints of the beams in different stories are different. Thus, single storey beam-column substructures may not represent the behavior of multi-storey frame exactly.

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