Experimental study of the horizontal progressive collapse of RC frames

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ABSTRACT

In reinforced concrete (RC) frames, horizontal progressive collapse (H-PC) may be triggered due to insufficient lateral stiffness of the structural systems. Current research studies have been mainly focused on vertical progressive collapse (V-PC) rather than H-PC. To analyze the effect of lateral stiffness and capacity on the behavior of progressive collapse (PC) of RC frames, a static collapse test of two asymmetric one-story beam-column substructure specimens with flange slabs was conducted. The two specimens S1 and S4 were subjected to an interior column removal scenario, in which S1 has one edge column and S4 has an edge and a penultimate column. The test results show that under the compressive arch action (CAA), S1 developed a smaller peak resistance and a larger vertical displacement as compared to S4 due to insufficient horizontal restraint of the columns. In addition, the contribution of the CAA within the beams and slabs in S1 was significantly reduced. Under the catenary action (CA), the collapse resistance of S4 developed rapidly and eventually exceeded the peak resistance of CAA; while the resistance of S1 remained almost constant, caused by the continuously weakened horizontal constraint provided by the edge column. Additionally, the final failure mode for both specimens was found to be in form of the compression-bending failure of the edge and penultimate columns.

Keyword: reinforced concrete frame structure; horizontal progressive collapse; static test; the effect of the lateral stiffness

INTRODUCTION

Progressive collapse (PC) of a building structure is defined as disproportionate collapse caused by an initial local failure due to accidents, such as gas explosion, bomb blast, vehicle impact and fire, which triggers and propagates chain reactions in the structural system (Ellingwood and Leyendecker 1978). Ever since the collapse event of Ronan Point apartment in UK in 1968,

in-depth understandings on progressive collapse were progressively studied from the basic concept (Ellingwood 2006) to structural system behavior (Yi et al. 2008). These studies included the experimental analysis of collapse mechanism (Yi et al. 2014), overall structure performance evaluations (Li et al. 2016), dynamic resistance and deformation demand theory (Li et al. 2014) and engineering design methods (Li et al. 2011), etc. However, current test and numerical studies on progressive collapse of reinforced concrete (RC) frames have been mainly focused on the cases where both beam ends were restrained and the collapse was confined within the restrained area. Further, the actual structural behavior is much more complicated and can be divided into six types (Starossek 2007) according to different causes and collapse characteristics, and the corresponding collapse mechanisms are different.

Progressive collapse failure can be divided into two types: vertical progressive collapse (V-PC) and horizontal progressive collapse (H-PC) in terms of the direction of propagation (see Figure 1).

Figure 1. Different collapse modes of frame structures

Figure 1- drawn by the authors

(1) When the adjacent substructure has sufficient lateral stiffness and bearing capacity, the internal forces generated by the initial local failure of the structure will not cause subsequent damage to the adjacent substructure, by which the progressive collapse only propagates in the vertical direction;

(2) When the adjacent substructure has insufficient lateral stiffness and bearing capacity, subsequent damage will occur in the remaining substructure. This in turn leads to propagation of the collapse not only in the vertical direction but also in the horizontal direction, which would eventually trigger a large scale collapse and overall damage of the structure.

Existing experiments investigating the progressive collapse of RC frame structures have been predominately centered on the V-PC mode. Only a few scholars began to notice different mechanical performance of the frame beams (or slabs) with inadequate lateral restraints (frame columns) (Pham and Tan 2015). In this study, two 1/3-scaled RC frame substructures with flange slabs were fabricated. Specifically, the two specimens S1 and S4 represent the interior column removal scenarios, in which S1 has one edge column and S4 has an edge and a penultimate column. Further, a static loading approach was used to investigate the mechanical

performance of H-PC by analyzing the development of the material damage and the collapse resistance of the specimens.

EXPERIMENTAL SCHEME

Design of the specimens

A 7-story 6×6 bay RC frame structure was designed according to the Chinese structural standards (MOHURD 2010), from which two single-story planar frames were isolated as the test substructures, enclosed by the red dash line rectangles in Figure 2a. The seismic design intensity is 8 degrees, i.e., the peak ground acceleration (PGA) is 0.2g for the design earthquake (with a 10% probability of exceedance in 50 years). 1/3 scaled specimens were tested according to the existing experiment (Yi et al. 2008), in which the beam and column sections were 100 × 250 mm and 250 × 250 mm, respectively. The thickness of the slab was 50 mm, the concrete cover of the beams and slabs was 8 mm and 7 mm, respectively. In addition, given that the horizontal tensile force generated in the slabs and beams is the main cause of the failure of the frame column, flange slabs with effective width were also fabricated in this work. The width was regulated by the code (MOHURD 2010).

Two specimens S1 and S4 were fabricated to investigate the influence of the lateral stiffness and bearing capacity of the surrounding structures on the collapse-resistance capacity of the whole structure, from which a interior column was removed, respectively, as seen in Figures 2b and 2c. Figure 2 also presents detailed dimensions and test setup of the specimens. Cross-sectional dimensions and reinforcement details of the beams and columns are given in Figure 3. The test frame and its foundation beam were casted with C50-grade and C30-grade concrete, respectively. The longitudinal reinforcement and the stirrup in the beams and columns adopted HRB335 and HPB300, respectively. HPB300 were also used to reinforce the flange slabs. Detailed material properties of reinforcement and concrete are summarized in Table 1.

(a) Prototype structure

Note: 1. Hydraulic jack; 2. Base plate; 3. LVDTs: 3-x; 4. Concrete strain gauge:4-x; 5. A~I: typical section.

Figure 2. Plan view of prototype building and corresponding test specimens S1 and S4(unit: mm)

Figure 3. Dimensions and reinforcement of cross-section of specimens (unit: mm)
Figure 2 & Figure 3 - designed by the authors

Table 1. Material properties of reinforcement and concrete

Specimen	Rebar type	f_y /MPa	f_u /MPa	E /GPa	δ (5d) /%	f_{cu} /MPa
S1	$\Phi 6$	394	519	220	20	29
S4	$\Phi 10$	419	617	219	18	45

Note: E , f_y , f_u , δ , f_{cu} are the elastic modulus, yield strength, ultimate strength, elongation and cubic compressive strength of concrete, respectively.

Test setup and instrumentation

Two and three jacks were set on top of the columns for S1 and S4, respectively. A load of 530 kN (the design axial compression ratio: 0.6) was applied on the edge and penultimate) columns (see Figures 2b and 2c) and remained unchanged throughout the test to simulate the gravity load. Subsequently, an increased displacement was applied on top of the removed column.

Linear variable differential transducers (LVDTs) were installed to monitor the vertical displacements of the beams and the horizontal displacements of the RC walls and the removed column, as shown in Figures 2b and 2c. Results of LVDT 3-6 indicate that no significant lateral displacement was developed in the RC walls which confirmed reliability of the horizontal

restraint. A series of steel strain gauges at key sections of the beams, slabs and column were arranged as shown in Figure 4.

Figure 4. Arrangement of strain gauges on reinforcing steel (unit: mm)

Figure 4 – designed and drawn by the authors

EXPERIMENTAL PHENOMENON

The load-displacement curves of the specimens (S1 and S4) are shown in Figure 5, represented by the applied load versus the vertical displacement of the removed column. The mechanical response of the specimens can be divided into two stages: compressive arch mechanism and catenary mechanism.

Compressive arch mechanism

In the stage of compressive arch action (CAA), the load was resisted by the flexural capacity and axial compressive force of the beam and slab in the specimens. It can be seen from Figure 5 that S4 exhibited a 17% larger collapse resistance (80kN) compared to that (69kN) of S1. This is because S4 has an extra bay in comparison to S1, thus the horizontal restraint of the beam and slab in S4 was greater. Note that the flexural bearing capacity at the beam ends of the two specimens was the same, and higher bearing capacity in S4 was provided by the greater axial compressive force developed through CAA. Quantitative evidence to support this statement is presented in Figure 6 where the average negative peak strain value of the bottom beam reinforcement in S4 is 34% larger than that in S1. Further, the top beam reinforcement in S1 was in tension. In addition, the stronger restraint also caused a smaller displacement in S4 corresponding to the peak value in CAA, which is 23% smaller compared to that in S1.

Figure 5. Load vs. mid-span deflection

Figure 6. Strain of reinforcing steel in the beams at section E of S1 and S4

Catenary mechanism

S1 and S4 underwent the transition from CAA to catenary mechanism (CA) at the displacement of 186mm and 148mm, respectively, as shown in Figure 5. The concrete in compression at the bottom of the edge column was found to have crushed at the displacement of 252mm, causing a decrease of the flexural capacity in the bottom column, which subsequently lead to a decline of the collapse resistance of the specimen. Within the displacement range from 186mm to 252mm, the bearing capacity of S1 increased slowly (from 46kN to the peak load of 49kN) since the horizontal restraint of the specimen was weakened. The bearing capacity in S4 increased significantly after entering the stage of CA, and the peak value (84kN) even exceeded that in CAA stage (81kN). The above phenomenon reflects that, in this stage, the large horizontal stiffness of S4 provided an adequate restraint for the development of catenary mechanism. On the contrary, due to the insufficient lateral restraint in S1, the axial internal force of rebars within the beam and slab were continuously released with increased displacement, making the rebars being unable to provide large bearing capacity for S1. The strain data of the rebars at section E in S1 and S4 also verified this phenomenon, where the strain values in S1 remained almost constant, while the strain values in S4 increased steadily after entering CA stage up until rebar fracture, as seen in Figure 7.

Figure 7. Strain of reinforcing steel in the slabs and beams at section E

Failure analysis of the edge and penultimate columns

It can be seen from Figure 8 that the horizontal displacement patterns of S1 and S4 are similar. The axial compressive force in frame beams pushed the edge and penultimate columns outwardly under small deformations and then pulled it inwardly under large deformations (the outward and inward horizontal displacements are negative and positive values, respectively, shown in Figure 8). In the CAA stage, the maximum value (3mm) of the outward lateral displacement of S1 was developed when the bearing capacity reached the peak value. For S4, the maximum outward displacement is only 0.1mm, which can be neglected. In the CA stage, the maximum values of the inward lateral displacement were measured as 93mm and 60mm in S1 and S4, respectively. As the stiffness of the column was enhanced in S4 by its extra bay, the lateral displacement in S4 is relatively smaller in comparison to that in S1 under both CAA and CA stages. Corresponding to the maximum lateral displacement, plastic hinges were formed at the bottom of the edge and penultimate columns.

Figure 8. Horizontal displacement of the top of the edge columns

CONCLUSION

The mechanism of horizontal progressive collapse and different horizontal restraint levels were studied by the experimental tests, the results show that:

- (1) In the stage of CAA, due to the smaller horizontal restraint, the peak resistance of S1

was 17% less than that of S4 and the corresponding displacement was 23% larger. The inadequacy of the horizontal restraint limited the development of the axial force in the beam, which reduced the contribution of the axial force to the collapse resistance.

(2) In the stage of CA, the collapse resistance of S4 developed rapidly and even exceeded the peak resistance of CAA, while the resistance of S1 remained almost constant. The axial force of rebars within the beam and slab were continuously released in CA, making the rebars being unable to provide large bearing capacity for S1.

(3) The ultimate failure mode for both S1 and S4 was in form of compression-bending failure in the bottom of the edge and penultimate columns caused by the loss of their stiffness and capacity.

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