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Experimental Research and Finite Element Analysis of Progressive Collapse Resistance of RC

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In this experimental study of continuous collapse of reinforced concrete frame structure, the authors design a 2*2 layer of reinforced concrete frame model and simulate the continuous collapse process of frame structure with the pseudo static test method, and hybrid control force and displacement loading method. While the Angle collapsed failure process and plastic hinge form development are observed, the date on resistance, reinforcement and concrete strain is collected and the characteristics of resistance to the continuous collapse of reinforced concrete frame structure is analyzed. By using ABAQUS finite element software to simulate the process of continuous collapse of reinforced concrete frame structures prism failure, resistance curves are spotted and are compared with the experiment result, and the congruency identified between them shows that the simulation can better simulate the process of continuous collapses of the model. On the basis of the corner posts failure simulation, the progressive collapse process is simulated and analyzed with ABAQUS when the reinforced concrete frame structural long side of the column are failure, and accordingly what is discussed is the collapse of the force during the conversion mechanism. The simulation results also show: Firstly, that during the conversion process of column failure, framework has gone through three stages ranging from the elastic stage, the stage of formation of plastic hinge, to the stage of concrete damage and suspension cable; Secondly, the beam in frame failure area has experienced the conversion process by bending to the suspension cable to provide continuous collapse resistance.

1. Introduction

The integral security of structure has been the chief problem of structural design. Research on structural progressive collapse enters into the sights of engineering since Ronan Poing apartment collapsed in 1968 (Ellingwood, 2006). Since then, the collapse of Alfred P. Murrah federal building (ASCE7-05, 2005) and WTO building (Mitchell and Cook, 1984) made the progressive collapse problems become the hot topics in the study of civil engineering. How to avoid the structural progressive collapse in terrorist attacks, explosion, fire and other emergencies caused by accidental loads has received considerable attention from engineering. Structural partial failure may cause the destruction in overall or a major part of the whole structure. To put it in another way, the scope and extent of final destruction is disproportionate with the scope and degree of initial destruction, and this kind of failure mode is called progressive collapse or disproportionate collapse in engineering. At present, many domestic and overseas scholars have done a large amount of research. Mitchell and Cook (1984) completed a set of scaled reinforced concrete floor collapsed destruction test, and obtained the analysis and calculation model of slab in collapsed state; Hadi and Alrudaini (2011) presents a new approach to prevent the progressive collapse of reinforced concrete buildings resulted from the potential failure of a column; Sasani et al. (2007) completed an oriented blasting demolition of the bottom's outside column experiment on a 10-layer reinforced concrete frame structure; Jinkoo and Taewan (2009) used the method of pushdown to analyze steel frame model to research the factors which influence the continuous collapse resistance ;British scholar Feng (2009) used the finite element software ABAQUS to simulate a bilaminar steel structure; Liu(2013) studied the dynamic characteristics of reinforced concrete members and used the numerical simulation to analyze the reinforced concrete members' dynamic experimental process. simulated the process of collapse under earthquake; Li et al. (2008) simulated a two acrosses three layers

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reinforced concrete frame by the explicit dynamic analysis software LS-DYNA; Li (2011) embed the THUFIBER program into MSC. MARE finite element software, which referred to analysis process in DOD(2005), to simulate a 8 layers reinforced concrete frame, which was designed according, to the specification in our country, it analyzed the continuous collapse resistance, and studied the design method of continuous collapse resistance of reinforced concrete frame systematically; Sasani M.(2014) through abstract experimental and analytical studies on a two-span fi xed-end RC beam; Yi et al. (2007) in Hunan university simulated the pseudo-static collapse test on a three-across four-layer reinforced concrete plane frame; Wu (2010) and Zou (2011) also used the method of quasi static loading to study resisting progressive collapse on a four-layer space frame without plate. Due to the complexity of mechanical mechanism of collapse process, although many scholars at home and abroad carried out a lot of works, but in view of the framework's continuous collapse, related research is still not perfect, it still needs to carry out the experimental study and finite element analysis.

Adopting reinforced concrete frame as the research object, the present study carries out progressive collapse test based on the case of corner column failure, observes the framework model's failure characteristics during progressive collapse and makes the simulation analysis of the failure and corner and middle column's failure process by employing ABAQUS.

2. The design and result of experiment

2.1 Specimen design

Test framework's size was 3.6 m * 2.6 m * 2.6 m. Longitudinal frame column grid layout spacing was 1.8 m, and the horizontal distance was 1.3 m. Column's section size was 133 mm * 133 mm, longitudinal beam section size was 67 mm * 150 mm, and the beam section size was 67 mm * 117 mm, with the thickness of the floor 30 mm. Height between the 1st layer and foundation beams was 1200 mm. Except for loading corner column, frame column of 1st layer reach out top pillar 50 mm. On the lower end of the model structure, the foundation beam was casted (base part did not cast plate). The test model is shown in Fig.1, board strain rosette's arrangement in Fig.2, the arrangement of concrete strain gauge in Fig.3. Strain gauge measuring steel main were put on the upper and lower side of the steel of the connection corner beam on the proximal and distal.

Figure 1: Model of frame structure

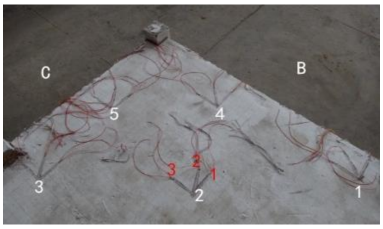


Figure 2: Layout diagram of strain rosette

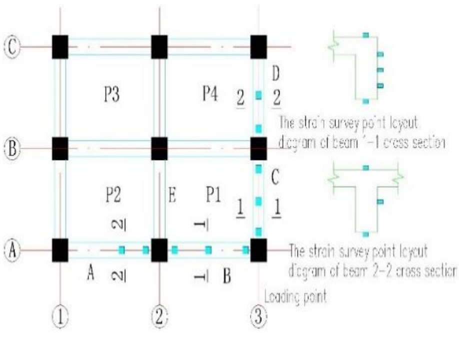


Figure 3: Layout diagram of strain gauges

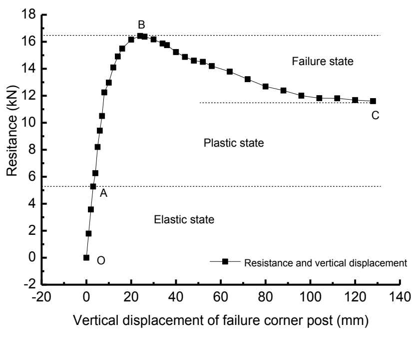


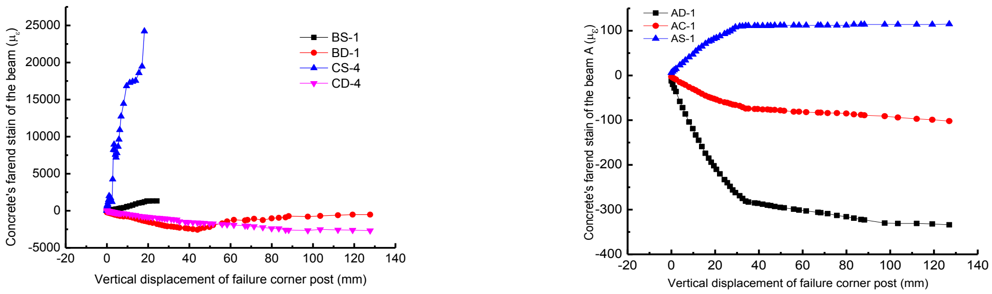
Figure 4: Vertical displacement curve

Force - displacement hybrid control loading method was adopted in the test. Loading system was: before the static test loading, the sensor signal acquisition channel was adjusted to zero. Force control gradation loading was adopted, and then the loading 1kN per time was increased through the hydraulic jack until the component generally yield, and then displacement control loading was started. During the displacement control loading, firstly the load was downward 5 mm at a time, and such a loading process was repeated for four times, and then the load was downward 10 mm at a time. After that, 10 minutes passed before data acquisition instrument was stable, and then data was read concerning compressive strain and tension strain of beam, plate and strain of steel bars and test section deflection value of concrete beam and plate. Along with the data reading, the value of force sensor synchronization was recorded at the same time. After completion of per level loading, maximum crack width was found with crack microscope and the range was measured with the steel rule, until the structure damage or bearing capacity fell sharply.

2.2 Result and analysis

Fig. 4 shows that with the increase in the vertical displacement, the whole collapse process roughly fell into three stages. 1) OA: elastic stage. Displacement increases with the increase in load, and displacement was 2.6 mm (point A) when the load was 5 kN, away from the Corner column, on the top of beam, crack signals the end of the elastic stage.2) AB: plastic hinge formation stage. With the increase in vertical displacement, tensile bars of beam end section yielded and plastic hinge formed gradually. When point B was reached, the vertical displacement was about 25 mm, with bearing capacity reaching the maximum value of 16.4 kN, and other sections of beam end connected with failure Corner column also successively forming plastic hinge. 3) The BC: failure stage. With the increase in vertical displacement, the plastic hinge region deformation increased sharply, and compression zone crushed and spalled locally. Due to the lack of the horizontal restraint of the frame column, beam can only rely on the bending bearing. When the vertical displacement reached 130 mm, the structure had been seriously deformed, and because the load of the failure pillars was transmitted by toughness of frame beam and bars in plate, we can think structure has collapsed.

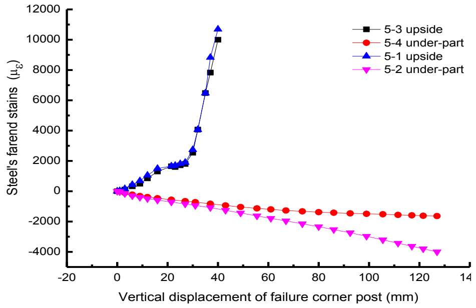
According to Fig.5 (a), at the beginning of loading, four parts of C beam were in tension. When the Corner column was displaced downward 2.6 mm, the concrete began to crack, showing that the elastic stress stage was over. As the corner column's vertical displacement continued to increase, tensile strain of one part in B beam increased correspondingly. When the vertical displacement of corner column was about 4.5 mm, this location found cracks, and then deformation gradually intensified, but due to the fact that the cracks went through the resistance strain gauge, the strain value was not an accurate reflection of the location of the real stress state. Down side of the concrete was at pressure initially. When the load was beyond the sustaining limit of the structure, then upper bars suffered serious deformation, plastic hinge was undermined, and concrete crushed, as a result, downside bars had the tendency of tension, the strain of concrete also gradually decreased, and it had the tendency of tension. Fig.5 (b) shows that with the increase in the vertical displacement corner column, concrete strain at upper A beam 1 was at tension, and lower and middle was at compression, indicating A beam has a tendency to move up, in line with displacement movement trend measured at A beam's point 4-5. Such a consistency implies that structure has a tendency to move to failure corner column from the other side.

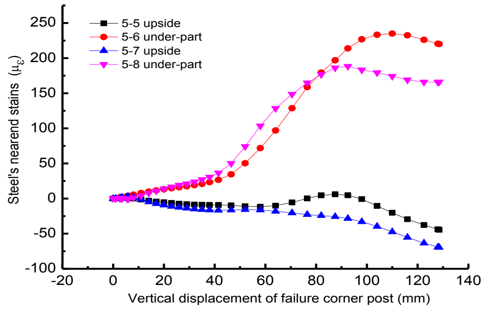


(a) Concrete stain of the farend on the beam B,C *(b) Concrete stain at position 1 of the beam A*

Figure 5: Concrete stain of the beam

On the basis of strain changes of bars in experiments, we can presume bar's yield order of frame beam section. According to the data measured, the upper side of bar 5-1 of 4 point in C beam was the first to yield, while at the same time the upper side of bar 5-3 of 1 point in B beam started to yield. The strain suffered by the two steels has been on the rise, in line with the actual working situation. Lower steel had been in a state of compression in the process of collapse, and the compressive strain increased with the increase in vertical displacement of failure column post. The 3 point's under-part tip bar 5-6 of tip beam C and 2 point's under-part tip bar 5-8 of beam B had been in a state of tension, while the upside tip bars 5-5,5-7 had been in compression state, and increased with the increase in vertical displacement, as shown in Fig.6.





(a) Steel's farend stains of beam *(b) Steel's nearend stains of beam*

Figure 6: Relationship between steel stains and vertical displacements of failure corner posts

3. Finite element analysis

3.1 Finite element analysis model

Finite element analysis software ABAQUS6.11 was used to establish finite element analysis model. In the process of simulation, this paper selected T3D2 on the part of rebar unit, and C3D8R unit on the part of concrete unit. Rebar used the reinforced double broken line model, without stiffness degradation and considering degradation of rebar in the process of collapse, density is 7800kg/m^3, modulus of elasticity, 2.08×10^{5}MPa, poisson's ratio is 0.3, yield stress 310MPa.

In this article concrete was under uniaxial compression, the tensile stress-strain curve followed the advised dimensionless form of "concrete structure design code"(GB50010-2010, 2010). In the simulation analysis, the concrete's damage indicators in the state of tension and compression are as follows:

$$d_t = 1 - \frac{\sigma_t E_c^{-1}}{\varepsilon_t^{pl}(1/b_t - 1) + \sigma_t E_c^{-1}} \quad d_c = 1 - \frac{\sigma_c E_c^{-1}}{\varepsilon_c^{pl}(1/b_c - 1) + \sigma_c E_c^{-1}} \tag{1}$$

Fig.7 shows that the resistance and displacement curve's rising period in simulation model is in good agreement with that in test, while decline period's overall trends are basically identical. Although it had a smaller limit value and a smaller displacement value compared with the experimental measured curve, it may be because the experiment has to do with the constitutive curves used in the finite element simulation model, boundary conditions and simplified model. The resistance value decreased gradually in the middle and late experiment, but remained invariant in the simulation. This is because stress reduced after reaching limit value in the test, but this was not considered in the simulation.

3.2 Side central column failure mode analysis

On the condition of the failure corner post, author simulated frame structural process of collapse when long side central column failed. Structural section size and reinforcement was identical with failure corner column.

Fig.8 shows that the curve is divided into four stages. OA is the elastic stage in which the structural resistance has a linear relationship with the vertical displacement of the disabled column, until cracks appeared at the concrete of under-part tip of lengthways beam; AB is plastic hinge formation stage. In this stage, the structural resistance increased gradually with the increase in the vertical displacement of disabled side central column. When the vertical displacement was 4.5 mm, rebars of under-part tip of lengthways beam, which was connected to the disabled central column, began to yield, and the structure began to form plastic hinge, until almost all steel within the beam yielded ,along with crushed concrete on local compression; BC is failure stage, in which compressed concrete began to be crushed locally, and concrete of plastic hinge region was cracking seriously when it arrived at the point of C; CD is suspension period. In this stage, the flexural bearing capacity of the frame beam was almost exhausted, Because structure depended on the beam to support outside load, the action of rebars within the beam began gradually to tension when it arrived at point D, and the vertical deflection of long beam span was 13.89%, far more than 10%. According to the DoD (2005) appendix B, we can think the structure has collapsed and lost bearing capacity.

Fig.9 illustrates in A, B beam, steel close to the bottom of the column yielded at the same time; rebars in the upside farend of beam E yielded firstly, and then rebars in the under-part nearend of beam E yielded while rebars in the upside nearend of failure center beam were compressed and tensile. Farend tensiled rebars of lengthways beam and compressed rebars of cross beam turned compression into tension.

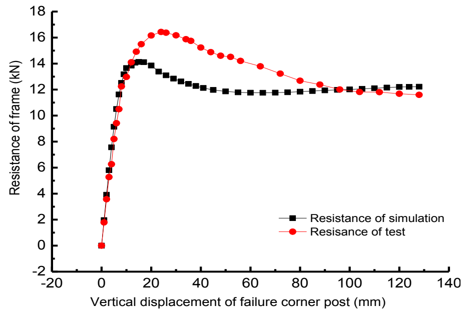


Figure 7: Comparisons between calculated and experimental

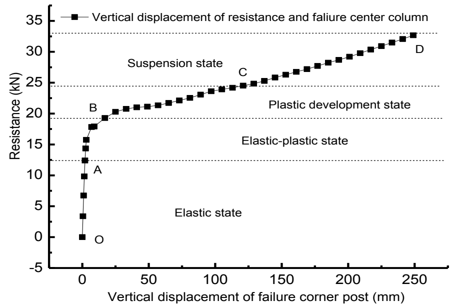
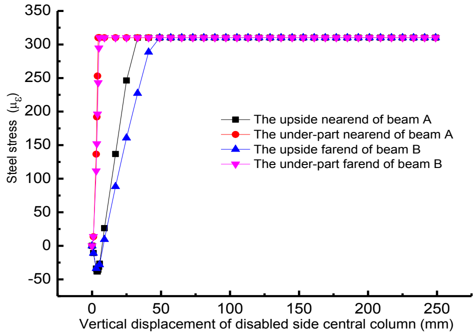
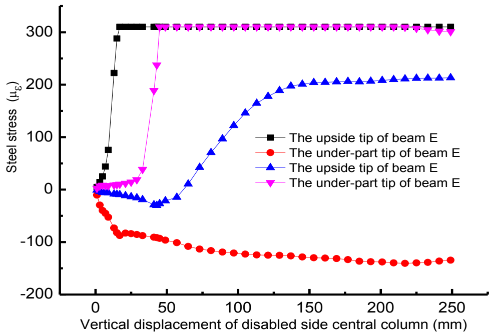


Figure 8: Resistance-displacement curve of calculated column



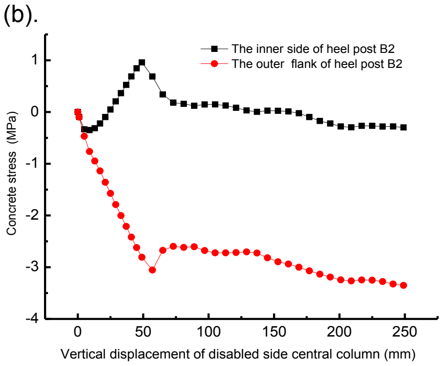
(a) Steel stress of nearend of lengthways beam



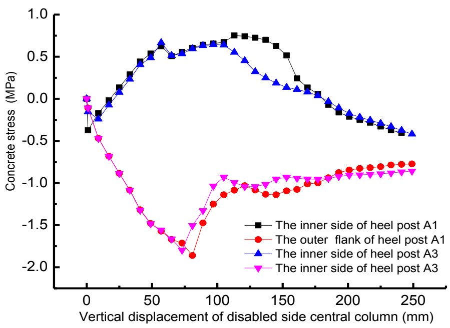
(b) Steel stress of transverse beam

Figure 9: Relationship between steel stress of frame beam and vertical displacement of side and center columns

At the beginning of the load, the axial compressive stress generated by load outside had the dominating effect. When vertical displacement of failure center column was 10 mm, the concrete compressive stress of B2 column foot inside reached the maximum; After that, bending stress generated by external load had the dominating effect, causing compression stress of B2 column feet inside begin to decrease and gradually become a pull stress, and compressive stress of column feet lateral increase gradually. This suggests that frame column was mainly axial deformation before point B. Capital load of failure column was passed to adjacent column by beam and slab, causing changes in adjacent column axial force. After the point B, with the development of the formation of a plastic body and the function of the suspension cable, the transverse B2 column, which was led by bending moments, had a larger deformation, as shown in Fig.10 (a). The column foot stress reached the maximum when vertical displacement was about 80mm, and then it began to decrease, showing that the framework had a tension and compression transformation in the process, which agrees with framework's rule: horizontal displacement was outward firstly and then inward, as shown in Fig.10 (b).



(a) Concrete stress of lateral column foot



(b) Concrete stress of lengthways column foot

Figure 10: Concrete stress curve of columns

4. Conclusions

(1) From the results it can be seen: frame model went through elastic stage, plastic stage and failure stage in the process of corner post's collapse; By studying collapse mechanism, it can be predicted that frame beams in the failure region used bending resistance to provide resistance of continuous collapse when corner posts were disabled; At the same time, it can be seen that in the process of load redistribution the failure column's load was transferred to columns on both sides by frame beams, which were connected with the columns. Its impact on the rest of the area was not big, and the size of the load transferring was associated with the stiffness of cross section.
(2) The continuous collapse process of failure corner posts was simulated by finite element analysis software ABAQUS6.11, and frame model's force-displacement curve agrees with the test curve, meaning it was reasonable to use ABAQUS for simulation; we can also use ABAQUS to simulate continuous collapse analysis of reinforced concrete frame.
(3) Continuous collapse process of frame model was simulated and analyzed when side central columns were disabled. From it we know the frame structure has undergone elastic stage, stage of formation of plastic hinge, concrete damage and suspension cable when side central columns were disabled, illustrating that frame beam in the failure area has experienced a transition process of providing resistance of continuous collapse, first bending resistance, then suspension cable.

Acknowledgments

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