Experimental Study on Collapse-Resistant Behavior of RC Beam-Column Sub-structure considering Catenary Action*

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ABSTRACT :

Under blast or impact loads, disproportionate collapse will be likely to occur for a majority of high-rise building without special collapse-resistant design. This paper presents a collapse-resistant experimental method and test results of five RC beam-column sub-structures with various arrangements of steel bars and various loading velocity. Both ends of the specimens were pin-connected and fixed on steel boxes and two steel tubes were embedded in both ends of the specimen. The load was continuously applied on the top of the middle column in scheduled time by the servo-hydraulic actuator with displacement-controlled mode, and the pin-supported RC beam-column experienced elastic, plastic and catenary stages. The collapse occurred at a vertical displacement of about 400mm due to the rupture of the bottom steel bars. By then, the member's ultimate capacity was about 2 times as much as the plastic stage's but the displacement was nearly 20 times.

KEY WORDS: collapse experiment, collapse-resistance, catenary action, collapse-resistant design

1. INTRODUCTION

Under blast or impact accidental loads, the failure of the main sub-structures of the frame will be likely to cause the disproportionate progressive collapse of the entire frame structures (Pearson1 2005, Corley 1998, Omika 2005) which, analytically, occurs when the structural load mode or boundary condition is changed and some structures fail because their load surpasses their bearing capacity. A series of terrorist attacks showed that a majority of casualties were directly resulted, from the structural collapse of the buildings, but not from the initial blast or impact, that's to say, the structural progressive collapse of the buildings dominated the casualties as compared with blast and its like.

The collapse-resistant design of the buildings generally refers to avoiding the progressive collapse which occurred when the buildings incur at blast or impact accidental events. A series of analytic and design methods were presented to the collapse-resistant design of the buildings (GSA 2003,DoD 2005,ASCE 2005) which were mainly divided into the indirect or direct design method(ASCE 2005). The former mostly set out the lowest requirements for the minimum resistance, ductility, continuity and integrity of the structures in view of their integrity under the restricted quantity of stories, height and span but found it impossible to be used for the future design though it might reduce the risk of structural progressive collapse ,while the latter dealt with the main structures of the directly bearing accidental load or the redistribution of the vertical applied load after they failed, and the member and the structure were designed so that their entire structures were capable of collapse-resistance. Many a literature emphasized the catenary action in keeping from structure collapse (Samuelt et al, 2003, Kapil et al, 2006, Yi et al, 2008). Samuelt's (et al, 2003) structural collapse-resistant experiment, i.e. , the simulated stationary load experiment of the failed column that one floor steel frame was equipped with reliable anchored cable under RC compound floor slab, proved the validity of the collapse-resistant measures. Kapil (et al, 2006) analyzed the catenary action in the collapse process of steel beam-column compound structures with the numerical method. Yi (et al, 2008) put forward that the ultimate collapse load capacity could be estimated with the method that the plastically calculated load capacity was about 68% as much as the ultimate catenary state and that the dynamic effect problem in the collapse process could be further considered with a simulated blast-resistant dynamic experiment, and he also analyzed the dynamic time history of the experimental frame with pseudo-static test results and the catenary action of the frame (He, et al. 2007).

* 基金项目：国家自然科学基金(编号:50678064)，长江学者和创新团队发展计划(批准号:IRT0619)

Research results (Samuelt et al ,2003, Yi et al 2008, Mehrdad et al, 2007) showed that the catenary action played an important relief and restrictive role in the progressive collapse process of the frame structures, which made the load-bearing capacity of the frame beam in an ultimate collapse state be much higher than that based on small deflection failure criteria and was the first defending frontline against the progressive collapse which occurred after the key supporting members failed under the accidental load, therefore it was meaningful to study catenary action mechanism and damage form. Here a continuous loading breaking experiment and study were performed on the five beam-column sub-structure specimens with various arrangements of steel ratio, grade, anchoring mode and loading velocity.

2. EXPERIMENTAL CONTENT

2.1. Specimen Design

Five beam-column compound specimens, 4400mm (L)×150mm (W)×300mm (H), were designed and made according to Chinese design code (GB50010-2002).The concrete strength grade was C30. Refer to Table 2 and Fig. 1 for the No. of specimen, steel ratio, grade and detail structural information of specimen. Properties of reinforcing steel and concrete are listed in Table 1. Two steel tubes, whose inside diameters were 80+0.1mm, were embedded in both ends of the specimens and connected with the hinge supports of the steel boxes with the help of steel pins of 80mm in diameter so as to ensure the supports' rotation.

The specimens were made by pouring with the special steel template cured by watering for 28 days at the room temperature and then installed for the experiment. The supports were fixed on two steel boxes, and then the boxes were fixed on the strong floor in laboratory.

Table 1 Properties of reinforcing steel and concrete

Material	Items		Measured values		
			HRB400	HRB335	HPB235
Steels	Yield strength/MPa		445	372	351
	Ultimate tensile strength/MPa		579	539	533
	Ratio of elongation	δ_5	29.7%	28.1%	26.1%
		δ_{10}	23.4%	22.5%	21.3%
Concrete(C30)	Cubic compressive strength/MPa		32		

Table 2 Detail information of designed specimens and load capacity

NO.	steel class	Steel ratio	Beam section	Ultimate load capacity (kN)		Maximum Displacement(mm)
				Hinge support	Simple support	
B2	HRB400	0.7%	Fig. 1(b)	113	37	372
B3	HRB400	1.4%	Fig. 1(a)	184	74	410
B4	HRB335	0.7%	Fig. 1(b)	99	31	421
B5	HPB235	0.7%		98	29	452
B6	HRB400	0.7%		115	37	393

Note: Ultimate load capacity of simple support is computed by the plastic methods, and others' are test value.

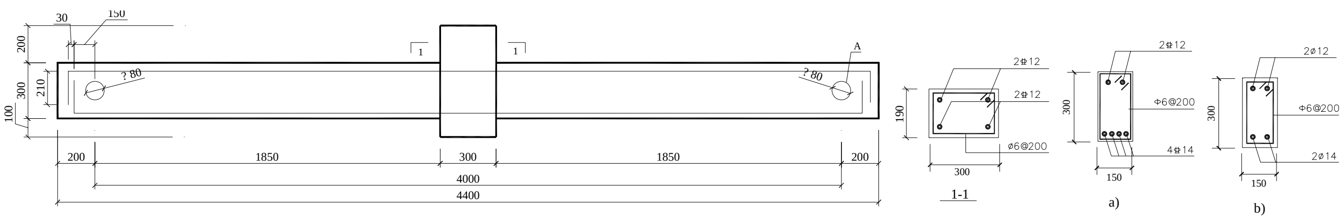


Fig. 1 Details of the specimen

2.2. Experimental method

At the beginning of the experiment, the jack which had been used for protection under the column was removed, the computer was used to control the servo-hydraulic actuator for continuous loading which moved from 0 to 450mm in scheduled time with the displacement controlled mode(Loading velocity of specimen B2 was 8mm/s, and others' 51mm/s), during which a MGC plus dynamic strainometer made by HBM Company was used to collect real-time dynamic data at a sampling frequency of 100Hz, which includes the applied load, displacement of the column, rotation at the end of specimen, steel strain and concrete strain.

3. TEST RESULTS

3.1. Relationship between the load and the displacement

Fig. 2 revealed the relationship curve between the load and the displacement of the column. Along with the increase of displacement, the beam mainly experienced the bending deformation in the elastic range and the concrete at the bottom of beam-end cross-section near the column broke with the increase of deformation. The neutral axis of the beam section moved upward. The whole specimen took on a form of arch mechanism. The arch thrust took place at the beam ends so that the load capacity of the beam-column sub-structures was increased. After bottom steel bars yielding, a turn point appeared in each curve, both the height and width of cracks increase rapidly and the concrete which could provide compressive stress was reduced. The specimen transferred from the arch action to the catenary action mechanism. Axial force in the beam is transferred from compression to tension. When the cracks developed to the whole section height, the applied load on the column of RC beam-column sub-structures was completely borne by the steel bars. And then the load-bearing capacity of specimen fell down dramatically, the structures were considered as completely losing the load-capacity and entering the collapse limit state. Fig.2 clearly showed the unsteady transition from the arch action to the catenary action. The process was very sensitive to the border condition and load condition.

The experimental curves in Fig.2 show that as the steel ratio increased, the ultimate load capacity of the specimens proportionally increased The deformation capacity was directly related to the ultimate load capacity and the steel grade. As the steel grade decreased, the ultimate deformation capacity of the specimens increased and the load capacity decreased, and the specimen reinforced with round steel bars were much better than the specimen reinforced with ribbed steel bars in forming the catenary action mechanism, since the round steel bars deformed more evenly and had larger elongation than the ribbed steel bars. Thus the resistance of the specimens at the catenary action stage was related not only to the strength of the steel bars but also to the larger deformation. It was known from each curve that the load-bearing capacity of the structures in the ultimate collapse state was about two times as much as that at the plastic stage, however, the deformation of the structures was nearly 20 times, which meant that the formed catenary mechanism of the structures was, in deed, beneficial to the collapse-resistance. Due to the rupture of the steel bars, the load capacity of the specimens fell dramatically and the structures entered the collapse status.

The rupture of the upper steel bars at the end of the specimens B3 occurred at the displacement of about 400mm, which meant that the only increase of the bottom steel ratio didn't contribute enormously to the development of catenary action and the upper steel ratio should be increased at the same time. As for the other specimens, the rupture of the bottom steel bars at the beam ends by the sides of the columns resulted in the collapse of the specimens. It was known from B3 and B6 in Fig.2 that the increase of steel ratio could only increase the collapse-resistant load capacity of the structures and could not increase the collapse-resistant deformation capacity. Representative rupture photos of the specimens were shown in Fig.3.

In summary, the catenary action could only play an important role in the increase of load-bearing capacity when quite a large deformation of the specimens took place. As far as the serviceability limited state design was concerned, this kind of large deformation would influence the normal use. However, after the key support member failed under the accidental extreme load action, the collapse breaking of the beam-column sub-structures was only required not to occur, i.e. , quite a large deformation of the beam-column sub-structures was allowed to occur. The test curve of load deformation showed that the flexibility increase of the beam-column sub-structures after they yielded and the large increase of load-bearing capacity due to the catenary action of the steel bars would not make the structures collapse at once. In this case it was acceptable to consider the active role of catenary effects in evaluating the load-bearing capacity in the collapse-resistant design of the RC beam-column sub-structures.

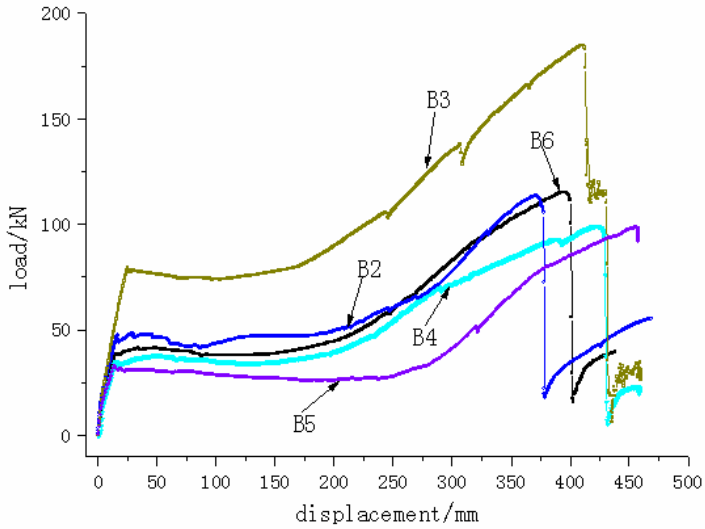
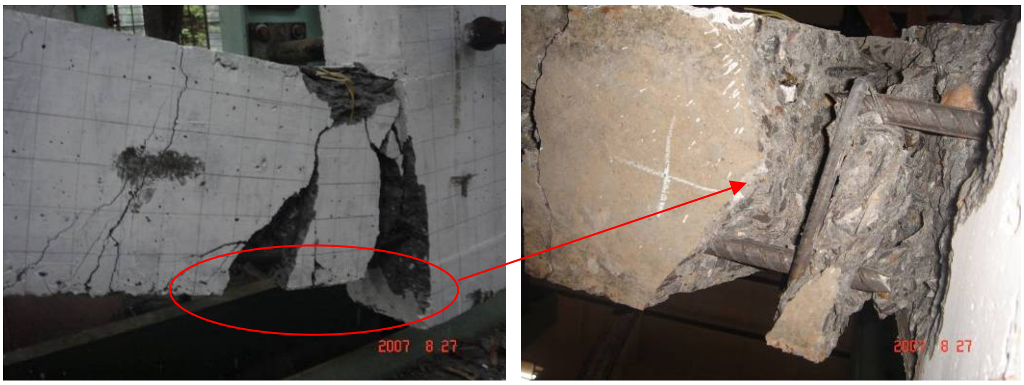


Fig.2 Load versus displacement curves

(a) Collapse ultimate state



（b) Bottom steel rupture of B4 near the column

Fig. 3 Representative Rupture photos of the specimens

3.2. Supports' Horizontal Displacement

Curves of the support's horizontal displacement were plotted in Fig.4, where the negative values meant outward movement while the positive values meant inward movement. It was known from the figure that the specimens moved outward at first and then begin to move inward at the displacement of about 100mm in the loading process. As the cracks of the specimens developed, the change of neutral axis resulted in the formation of axial compression in the beam, i.e., arch thrust action. As the deformation increased, the catenary action mechanism was gradually formed in structures and axial tension was induced. The larger horizontal force caused displacement at both hinge

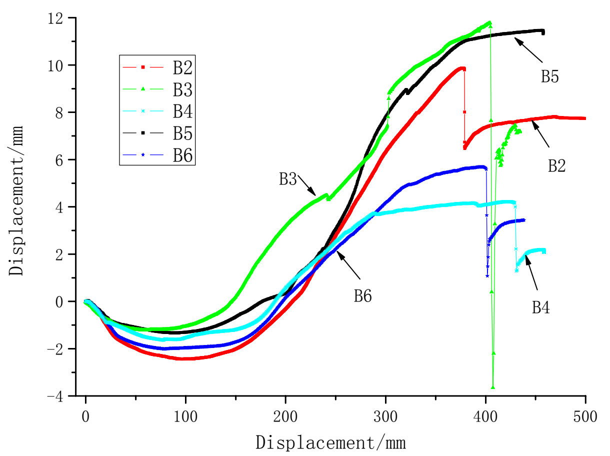
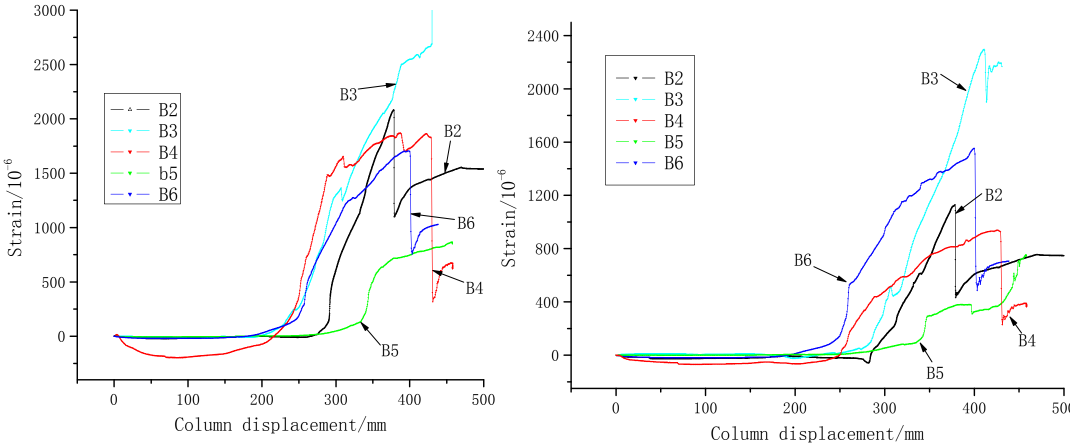


Fig. 4 Horizontal displacement curve of the support

supports.



(a) top steel (b) Bottom steel

Fig.5 Steel strain of beam section at the supports

3.3 Steel Strain of Beam Section at supports and crack development

The relationship between the steel strain of the beam section at the supports and the loading displacement was shown in Fig. 5. It was known from the curve that the beam section at the supports was first compressive and then tensile, which corresponded with the results shown in Fig. 2 and Fig.4. It was because the specimens took on arch action and catenary action along with the increase of the deformation. The strain's change of the top steel bars at the supports was larger than that of the bottom steel bars, which meant that the top tensile stress of the beam was larger than the bottom's, which could be proven by the vertical cracks that were found on the top of the beam at the supports.

In the whole loading process, the crack development was mainly divided into three stages. At the first stage of deformation, the loading displacement was within 200mm, the cracks mainly appeared in the section from the column to the 3/4 beam and gradually developed and widened from the bottom to top of the beam. The cracks at this stage were mainly resulted from the plastic deformation. The rest 3/4 beam section wholly rotated round the hinge supports. At the second stage, the main characteristic cracks appeared at the displacement of about 220mm, the diagonal cracks from the beam bottom to the beam top began to appear in the vicinity of the 3/4 section and the cracks which developed from the beam top to beam bottom appeared and gradually widened in the 1/4 cross-section at the displacement of about 250mm, the structures at this stage transformed from plastic deformation to the catenary action mechanism. At the third stage, the vertical cracks in equal width appeared in the 1/4 and 3/4 beam section, which was resulted from the axial strain of catenary action of the structures. The vertical cracks could be found on the top of the beam at the hinge supports after the experiment was finished.

4. APPLIED LOAD ANALYSIS OF BEAM-COLUMN SUB-STRUCTURES

The whole deformation process of the beam-column sub-structures mainly experienced three applied load stages, i.e., elastic deformation stage, plastic deformation stage and catenary action stage. As the displacement increased, the plastic hinges formed gradually at beam end sections near the column. When the whole beam sections entered the complete plastic state, the resistant moment reached the ultimate value and the plastic hinges were kept to rotate round the section. When the cracks went through the whole section, the applied load mechanism transformed to the catenary action. The applied load and the displacement passed on to the beam end through the steel bars. The catenary load could be worked out by the brief calculation diagram of the catenary action together with the rotary angle data obtained in the test (Yi, at al., 2008).

The beam-column structure was a substructure of the frame structures. In view of the member, if a beam-column sub-structure could be ensured that the entire structural collapse didn't occur after the frame column failed, it could reduce the incurred risk of disproportionate collapse of the entire structures. Therefore, as far as the collapse-resistant design was concerned, it was a key step to estimate the ultimate catenary action load of the beam-column sub-structures and the maximum deformation of the structures.

5. CONCLUSIONS

1. The catenary action could improve the beam-column sub-structures' load-bearing capacity and deformation capacity remarkably. And the tested specimens' ultimate load-bearing capacity was about 2 times as much as the plastic stage's but the displacement was nearly 20 times.
2. Steel anchorage measure and detail requirement of shearing resistance in concrete code are enough to form catenary action of beam-column sub-structure
3. The development of catenary action is mainly related to uniform elongation and strength of the steel, i.e. , uniform elongation avails to improve the maximum member's deformation and high strength the carrying capacity. So round bar is very good to the structural collapse-resistance.
4. With the increase of the deformation, beam-column sub-structure can form catenary action mechanism after arch action mechanism, which causes the axial tension after the axial compression during the process of the collapse.
5. The whole deformation process of the beam-column sub-structures mainly experienced elastic deformation stage, plastic deformation stage and catenary action stage. And maximum deformation and catenary force are two key design parameters based on catenary action mechanism.

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