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# Mechanism of Progressive Collapse Resistance of RC Frames Subjected to a Center Column Loss

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**Abstract.** Column loss scenarios are often used to investigate progressive collapse resistance of reinforced concrete (RC) buildings. The present study experimentally and computationally investigated the progressive collapse resistance of RC frames subjected to a center column loss. A one-third scale model of a single-story RC frame, which comprises two spans and two bays, was tested and computed. During the experiment and numerical analysis, the vertical displacement on the top of the removed center column was applied until it failed. This study gives insight into the progressive collapse behaviour as well as the failure mode of the RC frame. It has been shown that the catenary action can be activated in the frame beams, and the tensile membrane action can also be mobilized in the frame slabs at large deformations. Based on the experimental and computational results, the mechanism of progressive collapse resistance of RC frames was discussed. At large deformations, a simplified calculation method of progressive collapse resistance of RC frames was also proposed.

## 1. Introduction

Progressive collapse refers to a structure forms the local damage due to accidental events, then the initial local damage spreads from element to element, and eventually results in the collapse of an entire structure or a disproportionately large part of it [1]. Progressive collapse was first recognized as beginning with the collapse of the Ronan Point apartment building in 1968, and the September 11 Incident has once again aroused widespread attention to it. Over the past decades, various buildings around the world have been subjected to progressive collapse, which is caused by gas explosion, terror attack, or other factors. These progressive collapse accidents eventually resulted in a large number of casualties and property losses. Therefore, many countries have carried out research on progressive collapse resistance of structures and they have published some design codes, specifications and guidelines [2-3]. According to current codes, the resistance to progressive collapse can be achieved through maintaining the integrity and ductility of the structure, or providing alternative load paths, or providing sufficient tie force to critical structural members.

Current design specifications cannot completely satisfying the design requirements to resist progressive collapse. In order to better understand the mechanism of progressive collapse resistance of structures, further research is needed. Many efforts have recently been made to investigate the progressive collapse behavior of building structures under column loss conditions [4-8]. These studies pay more attention to the behavior of RC frame beams that bridge over removed RC columns in planar



frames or beam-column substructures. It can be noted that the greater catenary action can be activated in the frame beams at large deformations. Nevertheless, due to the presence of slabs, an actual floor structure in RC frames will not perform as isolated beams. Some researchers have examined progressive collapse behavior in beam-slab substructures or RC frames by experiments or numerical analyses [9-12]. Unfortunately, experimental and computational studies on progressive collapses of space RC frames is currently insufficient. In addition, the explanation of the mechanism of progressive collapse resistance is not clear and perfect enough.

The present study experimentally and computationally investigated the progressive collapse behavior of RC frames under a center column loss. A one-third scale model of a single-story RC frame was designed and tested. And a numerical analysis based on the LS-DYNA finite element software [13] was conducted to get more detailed structural information. At different progressive collapse stages, the further discussion on the mechanisms of progressive collapse resistance was provided. The contribution of the catenary action in beams and the tensile membrane action in slabs to the progressive collapse resistance of RC frames was evaluated.

## 2. Experiment scheme and finite element model

### 2.1. Experiment scheme

Loss of a penultimate-internal (PI) column on the ground floor is a critical internal column loss scenario, as shown in Figure 1a. The combined action of catenary forces in beams and tensile membrane forces in slabs at large deformation may cause the perimeter columns to fall inward, which triggers a horizontal and vertical mixing progressive collapse (Figure 1b). Consequently, the present study focused on the behavior of RC frame under PI column loss.

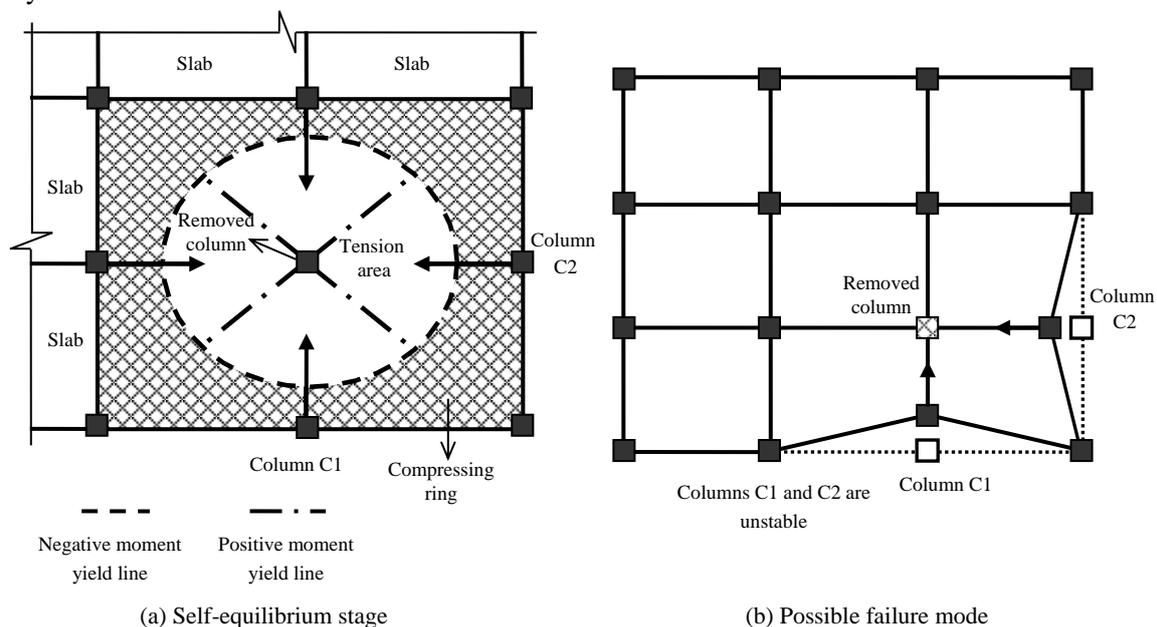


Figure 1. Structures subjected to a penultimate-internal column loss

According to the concrete design code and seismic design code of China [14-15], a prototype RC frame structure was designed, which comprises four spans, eight stories and four bays. Despite the different load and resistance factors, the Chinese code and ACI 318-08 [16] are generally similar. For the purpose of the progressive collapse experiment, a single-story one-third scale model was constructed, which is a segment of the prototype structure ground story and contains two spans and two bays. The height of the model frame layer is 1100 mm. The floor layout, reinforcement and cross-

sectional dimensions of the model frame see the literature [17]. The center column was removed in advance, but the adjacent frame joint was intact.

The instrumentation layout and test setup refer to the literature [17]. The model was vertically loaded on the top of center column that was removed by a MTS servo actuator. The loading was controlled by displacement and was 3 mm per minute. The model was loaded until it failed. At the ultimate state of progressive collapse, the steel bars in the frame beams fractured and the progressive collapse resistance reached its maximum.

## 2.2. Finite element model

To get more detailed structural information during the progressive collapse, a numerical investigation on the behaviour of the progressive collapse of the model frame was conducted by LS-DYNA. The analysis accounted for both material and geometrical nonlinearities, which included fractures represented as element erosion. In the finite element model, concrete and reinforcement were respectively constructed by solid elements and beam elements, which were finely meshed. The interface between concrete and reinforcement was simulated by the one-dimensional contact interface (Contact\_1d in LS-DYNA). The parameters of one-dimensional contact model refer to the literature [18]. The material models of concrete and reinforcement were selected the continuous-surface-cap model (Material 159 in LS-DYNA) and the piecewise-linear-plasticity model (Material 24 in LS-DYNA), respectively. And the key material performance parameters were obtained by the material experiments (see the literature [17]). Considering symmetry, only a quarter of the model was built in the numerical analysis, as illustrated in Figure 2. In the numerical investigation, the loading was exactly the same as that in the experiment.

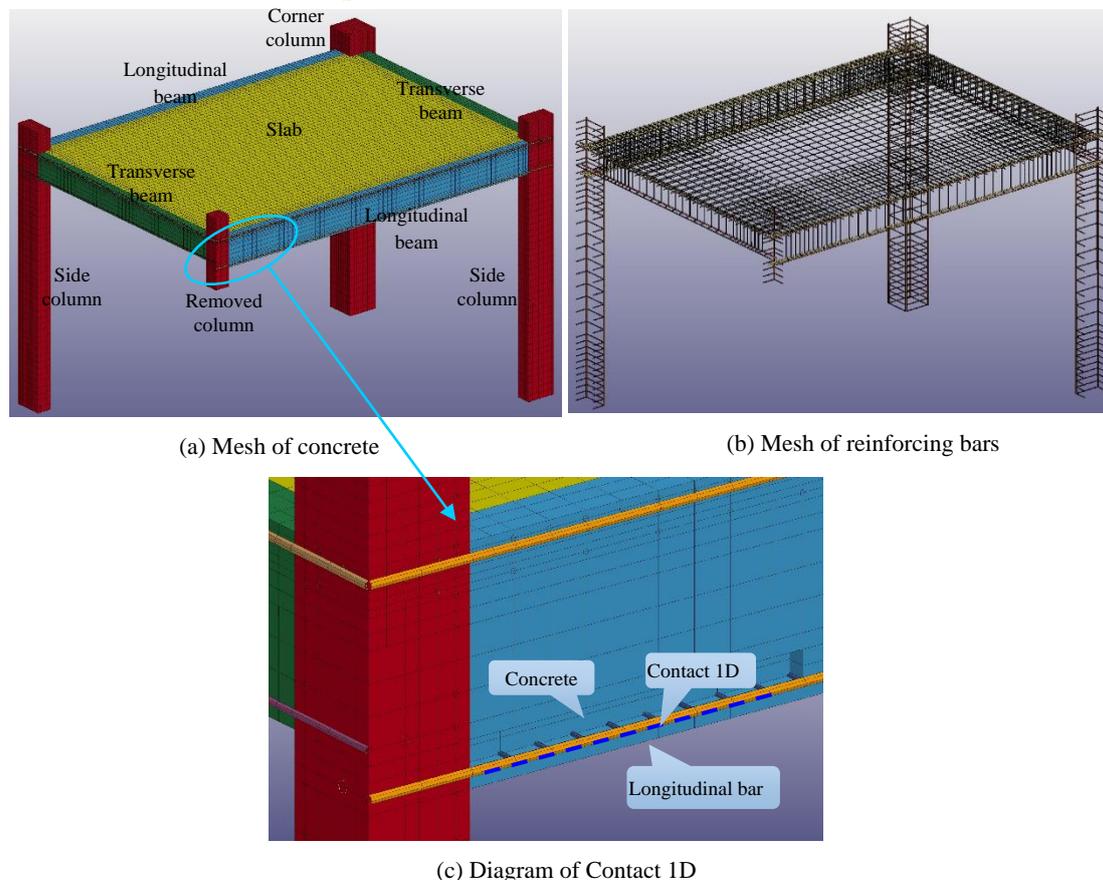


Figure 2. Finite element quarter-model of the experimental model frame

### 3. Progressive collapse process

The relation of the progressive resistance versus the vertical displacement on the top of the removed center column is shown in Figure 3. It can be found that the progressive resistance increases with the increase of the vertical displacement on the top of the removed center column. The stress of the longitudinal reinforcement at frame beam ends adjacent to the removed column is illustrated in Figure 4. Figure 5 presents the stress of the reinforcement at slab bottom in transverse direction. The stress results of the reinforcements are derived from the numerical analysis. According to the structural behaviour characteristics and the resistance-displacement relationship, the process of progressive collapse of model frame can be divided into three stages: the elastic stage, the elastoplastic stage and the composite stage of catenary action and tensile membrane action.

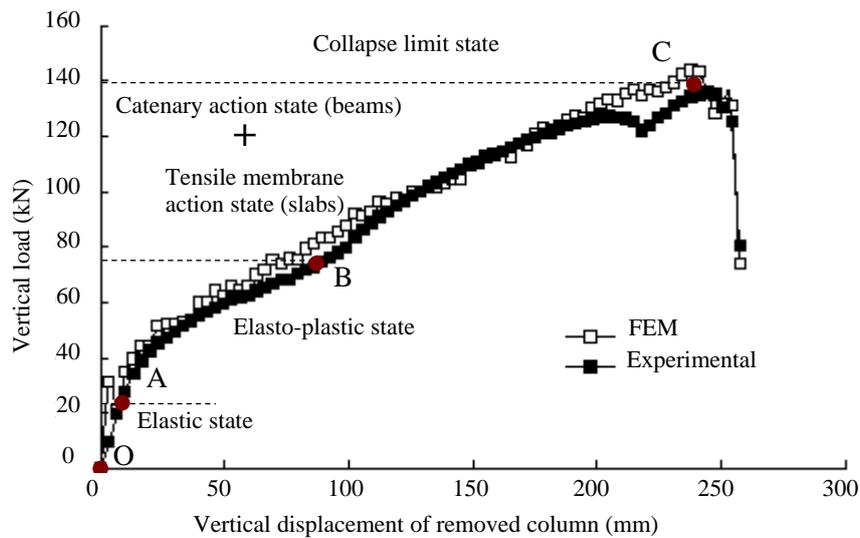


Figure 3. Removed column load versus vertical displacement of removed column

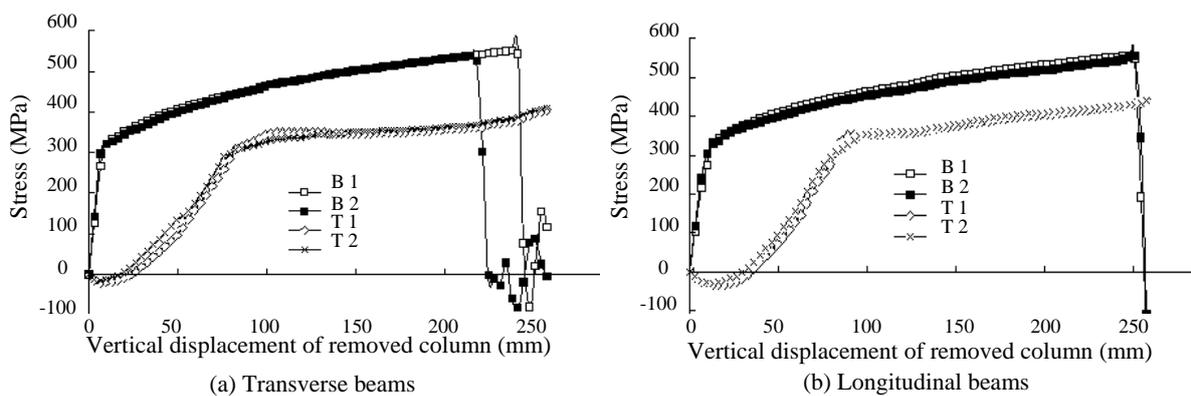


Figure 4. Longitudinal reinforcement stress at beam ends adjacent to removed column (B: bottom reinforcement; T: top reinforcement)

#### 1) Elastic stage

Section OA (see Figure 3) is the elastic stage, in which the resistance and the displacement are approximately linear. At State A, the cracks were observed at the bottoms of frame beams and slabs. In the elastic stage, the displacement on the top of the removed column is less than 8 mm. As shown in Figure 4, the bottom longitudinal reinforcement of frame beams is in tension, and the stress linearly increases with the increase of the displacement on the top of the removed column. In addition, the stress is much less than its yield stress. The top longitudinal reinforcement of frame beams is in compression, and the stress is very small in the elastic stage. In Figure 5, the stress of reinforcement at

the bottom of slabs linearly increases with the increase of the displacement on the top of the removed column, and the increasing rate increases with the decrease of the distance from the removed column in this stage.

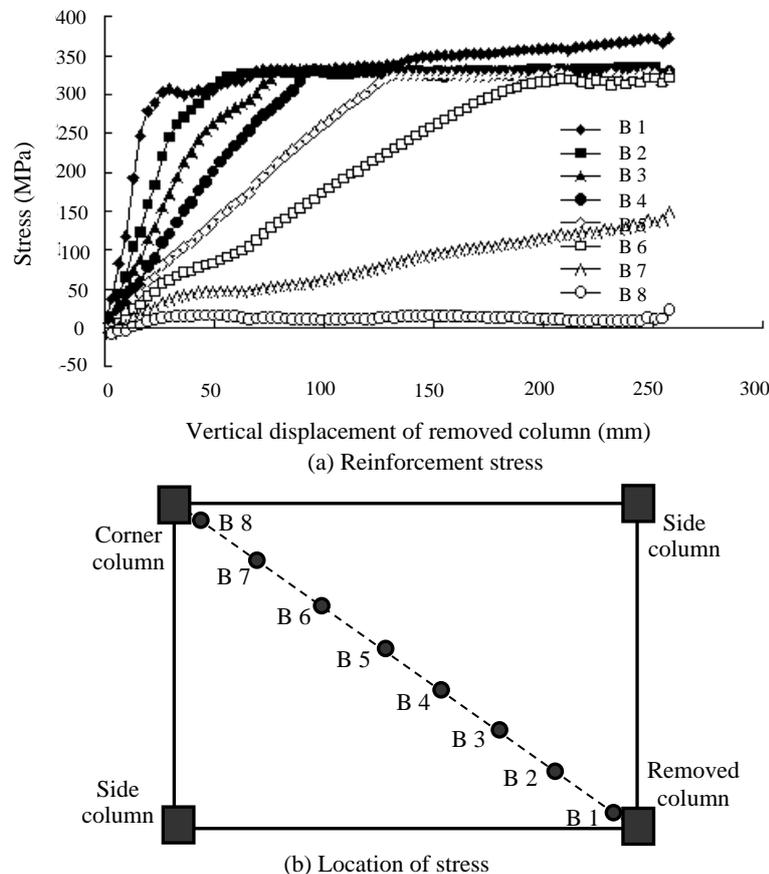


Figure 5. Slab bottom reinforcement stress in transverse direction

## 2) Elastoplastic stage

As illustrated in Figure 3, Section AB is considered as the elastoplastic stage. The resistance increases nonlinearly with the increase of the displacement on the top of the removed column. At State B, the displacement on the top of the removed column is approximately 85 mm. As observed in Figure 4, the bottom longitudinal reinforcement of frame beams enters the yield stage, and the top longitudinal reinforcement changes to tension from compression. As shown in Figure 5, a part of the bottom reinforcement of frame slabs near the removed column has reached the yield state, but the other part away from the removed column is still in elastic state. From the experimental results, it can be found that the clear cracks are found in the concrete of frame beams and slabs, and the plastic hinges and the plastic hinge lines are formed in the frame beams and slabs, respectively.

## 3) Composite stage of catenary and tensile membrane

Section BC (see Figure 3) is the composite stage of catenary action and tensile membrane action. In this stage, the tension cracks in concrete of frame beams and slabs have developed to penetrate the compressive zones, and the contribution of the plastic hinges of frame beams and the plastic hinge lines of frame slabs to the progressive collapse resistance, has become very small, which can be ignored in theoretical analysis. In Figure 4, it can be noted that the bottom longitudinal reinforcement of frame beams has entered the strain-hardening phase. The top longitudinal reinforcement of frame beams is fully in tension. As shown in Figure 5, most of the reinforcement at the bottom of frame slabs adjacent to the removed column, has entered the yield phase. A maximum resistance of 135.7 kN corresponding to a displacement of 256 mm can be observed at State C. In the limit state, the complete

fracture of one of the bottom longitudinal reinforcement of frame beams was found in the experiment and numerical analysis, as observed in Figure 6.



Figure 6. Rupture of steel bar in 2-axis frame beam

#### 4. Mechanism of progressive collapse resistance

In a center column loss scenario, because the spans of frame beams and slabs adjacent to the removed column become larger, they become the weakest part of the frame structures. Consequently, the catenary action of frame beams and the tensile membrane action of frame slabs can be considered as the main anti-collapse mechanism for a RC frame in the limit state.

##### 1) Catenary action of beams

Figure 7 presents the failure mode of the model structure. As depicted in Figure 8 (obtained from the numerical analysis) and observed in the experiment, it can be noted that the axes of frame beams remain straight in the limit state. Therefore, a model for the catenary action of frame beams is demonstrated in Figure 9. And the resistance of frame beams based on the catenary mechanism can be expressed as [8][19]

$$P_{ub} = \frac{(L_1 + L_2)v_u}{L_1 L_2} \cdot (A_{th}) f_y \quad (1)$$

where  $v_u$  is the displacement on the top of the removed column,  $A_{th}$  is the cross-section area of the longitudinal reinforcement through whole span,  $f_y$  is the yield stress of the longitudinal reinforcement.

##### 2) Tensile membrane action of slabs

The part enclosed by the negative moment yield lines is taken as the object of analysis. As illustrated in Figure 7, the negative moment yield lines approximately constitute an ellipse. Based on the principle of force equivalence, the boundaries of the analytical object can be assumed to be rectangular. Consequently, the model for the tensile membrane action of frame slabs is illustrated in Figure 10. Based on the analysis in Section 3, the boundaries of the analytical object can only resist pulling force, as shown in Figure 10.

Figure 11 shows the displacement at different positions in the longitudinal and transverse span centers of frame slabs in the limit state, which is derived from the numerical analysis. It can be observed there are small changes in the displacement in Section  $OA_1$  and Section  $OB_1$  compared to Section  $OA_2$  and Section  $OB_2$ . Therefore, based on the compatibility of displacement of frame beams and slabs, and Figures 8 and 11, the frame slabs GHK, HIK, IJK and GJK (Figure 10), which are enclosed by the positive and negative moment yield lines, can still be approximately assumed to be plane in the limit state.

According to the analysis of vertical force balance, the resistance of frame slab GHK based on the tensile membrane mechanism can be given as

$$R_{sGHK}^{tm} = (F_{y1}l_{x1} + F_{y2}l_{x2}) \frac{v}{\sqrt{v^2 + l_{y1}^2}} \quad (2)$$

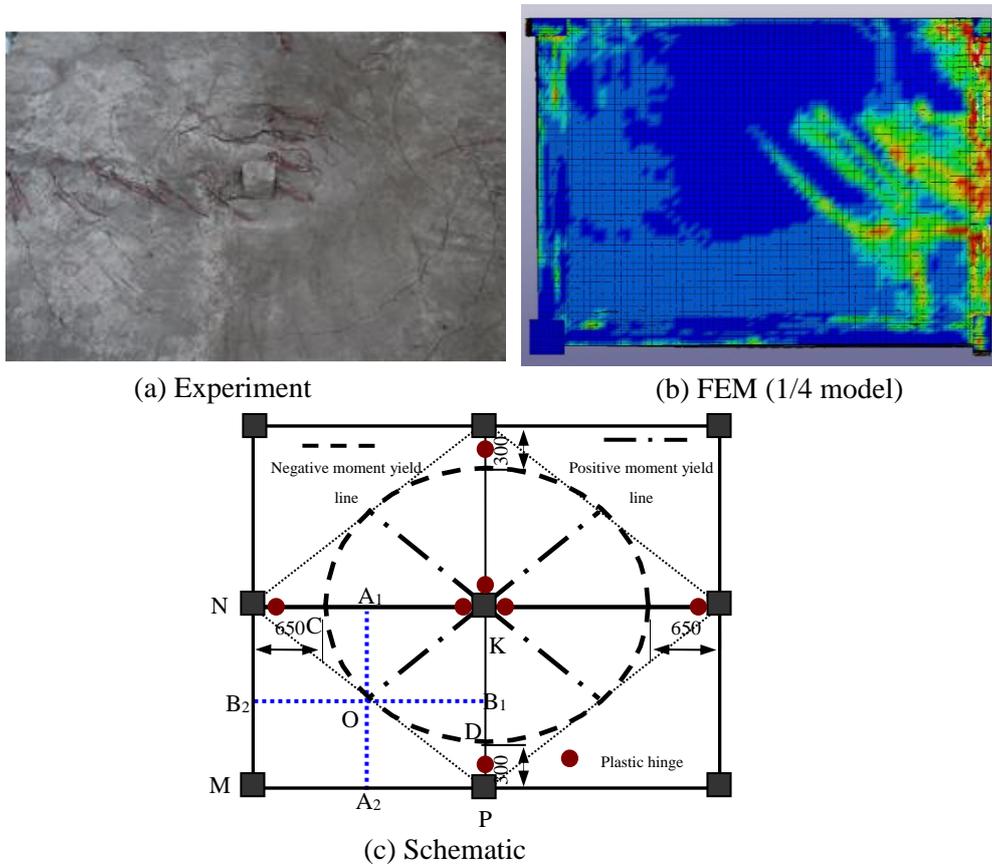


Figure 7. Failure mode of model frame (unit: mm)

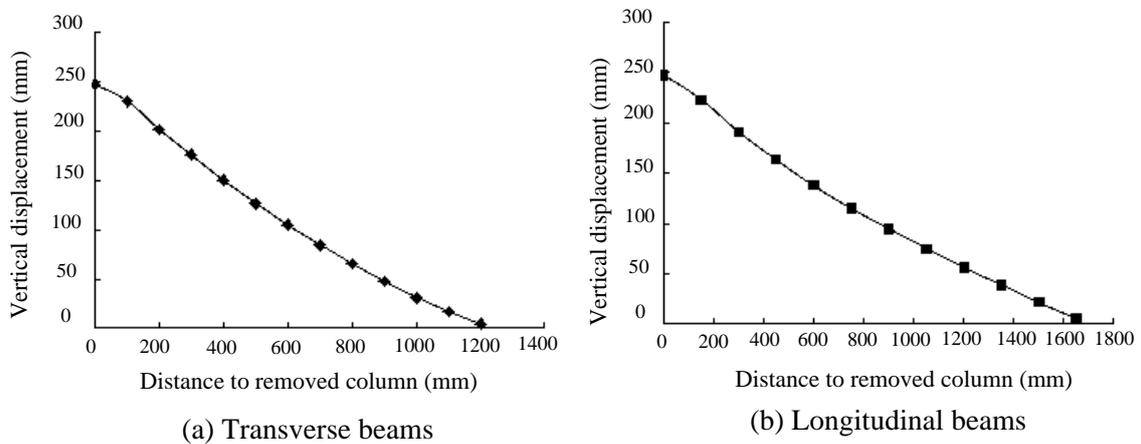


Figure 8. Displacement of different positions on frame beams in limit state

where  $F_{y1}$  and  $F_{y2}$  are the yield bearing capacities of the reinforcement within the unit width slab ① and slab ②, respectively;  $l_{x1}$ ,  $l_{x2}$ ,  $l_{y1}$  and  $l_{y2}$  equal the projection lengths of the negative moment yield lines in the corresponding axes, respectively;  $v$  is the displacement of Point K.

Once again, in slabs HIK, IJK and GJK, the resistance based on the tensile membrane mechanism can be obtained as

$$R_{sHIK}^m = \left( F_{x2}l_{y1} + F_{x4}l_{y2} \right) \frac{v}{\sqrt{v^2 + l_{x2}^2}} \quad (3)$$

$$R_{sIJK}^{tm} = (F_{y3}l_{x1} + F_{y4}l_{x2}) \frac{v}{\sqrt{v^2 + l_{y2}^2}} \tag{4}$$

$$R_{sGJK}^{tm} = (F_{x1}l_{y1} + F_{x3}l_{y2}) \frac{v}{\sqrt{v^2 + l_{x1}^2}} \tag{5}$$

According to the principle of superposition, the resistance of the whole frame slab GHIJ based on the tensile membrane mechanism can be expressed as

$$P_{us} = R_{sGHK}^{tm} + R_{sHIK}^{tm} + R_{sIJK}^{tm} + R_{sGJK}^{tm} \tag{6}$$

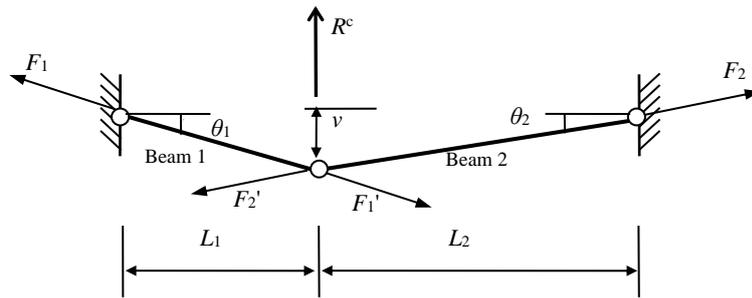


Figure 9. Catenary mechanism of frame beams

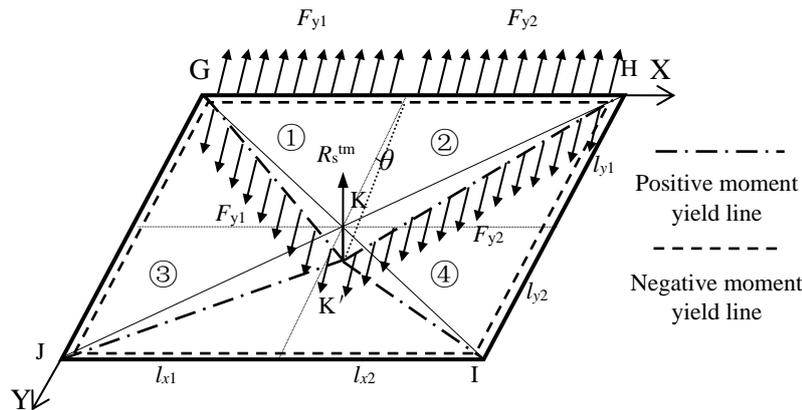


Figure 10. Tensile membrane mechanism of frame slabs

### 3) Validation

According to the experiment and numerical analysis, it can be found that the maximum displacement corresponding to the collapse limit state is dominated by the frame beams on the 2-axis (namely the short axis though the removed column). The maximum displacement on the top of the removed column can be obtained to be 261.7 mm based on the recommended method in the document [8]. Combining the maximum displacement, the geometric dimensions and the material parameters, based on the Equation (1), the collapse resistance of frame beams ( $P_{ub}$ ) can be obtained to be 62.5 kN.

Based on the compatibility of the displacement of frame beams and slabs, the displacement of Point K ( $v$ ) equals the displacement on the top of the removed column ( $v_u$ ) in the limit state. Consequently, the maximum displacement of Point K is also 261.7 mm. Once again, based on the Equations (2) to (6), the collapse resistance of frame slabs ( $P_{us}$ ) can be calculated to be 69.1 kN.

According to the principle of superposition, the total collapse resistance of the model frame is 131.6 kN, which is 3.0% smaller than that of the experimental results. In addition, comparing the collapse resistance of frame beams and slabs, it can be found that the latter is larger than the former.

This shows that the slab has a significant contribution to the collapse resistance of the space frame structures.

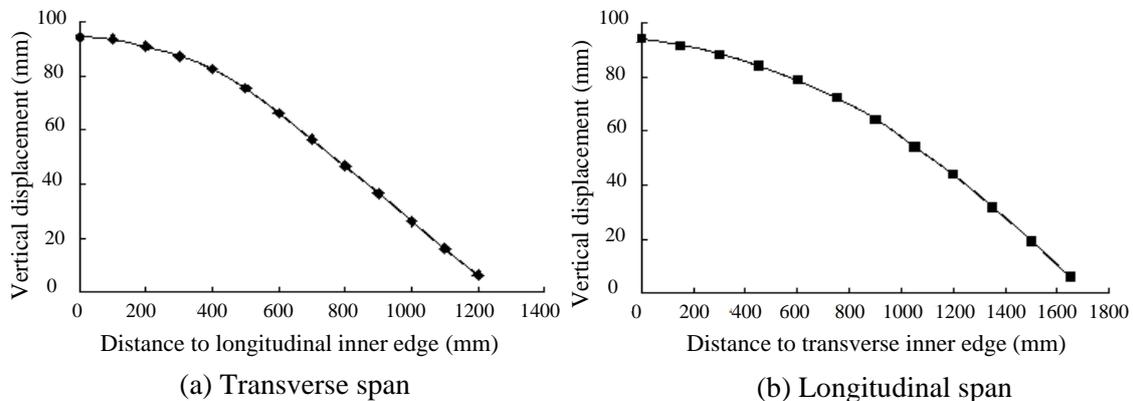


Figure 11. Displacement of different positions on frame slabs in limit state

## 5. Conclusions

The present study carried out the computational and experimental analysis for investigating the mechanisms of the progressive collapse resistance of RC frames subjected to a center column loss. According to the structural behaviour characteristics and the resistance-displacement relationship, the process of progressive collapse of model frame can be divided into three stages: the elastic stage, the elastoplastic stage and the composite stage of catenary action and tensile membrane action. The catenary action of frame beams and the tensile membrane action of frame slabs can be considered as the main anti-collapse mechanism for a RC frame in the limit state. At large deformations, a simplified calculation method of progressive collapse resistance of RC frames was also proposed, which has been verified by both experimental and numerical results.

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