Experimental and Computational Assessments of Progressive Collapse

Resistance of Reinforced Concrete Planar Frames Subjected to Penultimate Column Removal Scenario

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Abstract

Existing research studies have primarily examined the progressive collapse of frame structures under an inner column removal scenario. However, progressive collapse risk is much higher when penultimate columns close to the structural periphery are damaged due to weaker horizontal constraints. A static progressive collapse test was thus conducted in this study on two single-story beam-column planar substructures with flange slabs, in which a penultimate and an inner column were removed respectively. Compared to the specimen with an inner column removal, the specimen with a penultimate column removal exhibited a larger vertical displacement under the small deformation stage, which further reduced the
contribution of the compressive arch action to the collapse resistance. Under the large
deformation stage, the resistance of the specimen with an inner column removal increased
significantly, while that with a penultimate column removal was not enhanced notably
because the horizontal movement of its edge column resulted in a smaller rise of steel
strains under the catenary action. The internal forces were calculated using the measured
strain data at the key sections of the slab-flange beams. The calculated results also confirm
that the compressive arch action and catenary action were unable to be fully developed in
the specimen with a missing penultimate column. Finally, the outcome of the vulnerability
assessment of the prototype reinforced concrete frame reveals that there might be a
potential risk of progressive collapse to the structure under large deformations when a
penultimate column on the ground floor is damaged and the risk is higher when a
penultimate column on the top floor is damaged.

Keywords: Reinforced concrete frame; Progressive collapse; Penultimate column removal
scenario; Static test; Internal force calculation; Vulnerability assessment.

Introduction

Progressive collapse of a building structure is defined as the disproportionate collapse
caused by an initial local failure due to accidents that propagates and triggers chain
reactions in the structural system (Ellingwood 2006). In some countries, the design
regulations for progressive collapse prevention are specified in either the design codes (BSI
2002; CEN 2006; MOHURD 2010a) or the specialist guidelines (GSA 2013; DoD 2016). Design
methodologies reinforced concrete (RC) frame structures indicate that the compressive arch
action (CAA) at small deformations and the catenary action (CA) under large deformations,
provided by beams and slabs, can significantly contribute to the structural progressive
collapse resistance. With the presence of CAA and CA, large axial compressive and tensile forces are developed in beams and slabs. This implies that both ends of the beams and slabs should be strongly restrained to provide adequate reaction forces for balancing the internal axial forces.

Over the past decades, progressive collapse studies of RC frame structures have become increasingly popular owing to the advancement of the experimental and numerical techniques. In general, four major aspects have been investigated to date, including those related to the progressive collapse mechanisms (Kang and Tan 2017; Li and Sasani 2015), progressive collapse resistance assessments (Fascetti et al. 2015; Qian and Li 2015), dynamic responses of progressive collapse (Pham and Tan 2017; Peng et al. 2018), and relevant design theories and methodologies (Izzuddin et al. 2008; Li et al. 2011). Most studies have analyzed the effects of damage, failure, and fracture of materials and members on the structural resistance. The majority of tested RC frame specimens reported in the literature were set up with strong boundary constraints, representing the case of substructures being subjected to an inner column removal (ICR) scenario, such as published tests involving frame beams (Su et al. 2009), beam-column substructures (Sadek et al. 2011), beam-slab substructures (Ren et al. 2016), a multistory planar frame (Yi et al. 2008), and a real building (Sasani et al. 2007). Among these tests, the beam ends and the slab edges were strongly restrained by fixed boundaries, enabling the CAA and CA to be well developed.

Given that frame structures have insufficient lateral capacities to develop catenary actions when their corner or penultimate supporting members are damaged, enhancing local resistance of these areas is particularly required in the Unified Facilities Criteria (DoD 2016) to resist overall progressive collapse (Stevens et al. 2011). Currently, only a small number of
experimental studies have been conducted on the RC substructures with insufficient horizontal constraints (Yu and Tan 2013; Choi and Kim 2011; Lim et al. 2017). Yu and Tan (2013) indicated that the CAA is significantly affected by horizontal constraints in that higher constraints can lead to earlier mobilization of CA. Lim et al. (2017) also revealed the adverse effect of inadequate horizontal constraints on CA under penultimate column removal scenarios. Although issues pertinent to the insufficient boundary constraints have been raised in the literature, no detailed design requirements are available in the existing codes and guidelines. It should be noted that upon damage of a penultimate column (adjacent to the edge column), the upper frames can deform downward. Under this scenario, only the bottom of the edge column remains horizontally restrained. For this reason, the vulnerability of the frame subject to a penultimate column removal scenario is also necessary to assess for the purpose of achieving safe designs. Given limited work dealing with insufficient horizontal constraints, further in-depth studies are required to explore the collapse mechanisms of RC frames under such a restraint condition.

In this paper, a planar beam-column substructure with flange slabs, i.e., Specimen PCR, was designed, in which a penultimate column was removed, and thus a weaker horizontal constraint was provided to the slab-flange beams by only a single edge column on one side of the specimen. In addition, Specimen ICR, with an inner column being removed, was also designed with identical reinforcement details. Serving as a reference specimen, ICR had a strong horizontal constraint to the slab-flange beams provided by both sides of the specimen. Both specimens were tested to large deformations. Their progressive collapse mechanisms were investigated by analyzing their resistances, damage patterns, and deformation modes, as well as calculating the internal forces using the strain data at key sections. The
vulnerability of the prototype RC frame subjected to a penultimate column removal scenario was also assessed through computation.

**Experimental Scheme**

**Design of Specimens**

A 7-story prototype RC frame structure was designed according to the Chinese design codes (MOHURD 2010a, b). The design dead and live loads for the floor were 5.0 and 2.0 kN/m², respectively; and those for the roof were 7.5 and 0.5 kN/m², respectively. The seismic intensity for design was set as 8°; i.e., the peak ground acceleration was 0.2g for the design earthquake (with a 10% probability of exceedance in 50 years), where g is the acceleration of gravity. Similar to the existing progressive collapse tests (Eren et al. 2019; Khorsandnia et al. 2017), a planar frame on the ground story being isolated from the prototype structure was taken as the substructure to be tested and the tested frame was scaled down to 1/3 of the original dimension (Fig. 1). Published literature confirmed that the critical scaling factor for RC specimens not damaging in shear is 1/4, with which the resistance mechanisms and load-displacement relations can be well represented (Abrams 1987). Indeed, 1/4 (Pham and Tan 2013; Qian et al. 2014), 1/3 (Qian and Li 2015; Yi et al. 2008), and 1/2 (Yu and Tan 2012) scaling factors were adopted in many progressive collapse tests of RC structure and substructures. Two substructure specimens were fabricated to create a penultimate column removal (PCR) and an inner column removal (ICR) scenario (Fig. 2). In Specimen PCR, the horizontal constraint was provided by only a single edge column at one end in order to explore the progressive collapse mechanism triggered by an insufficient horizontal restraint. To save laboratory space, the frame component with two or more bays in both specimens was simplified by a one-bay RC frame with a concrete infilled wall. A numerical simulation
was conducted to confirm that the lateral stiffness of the left-hand four-bay RC frame adjacent to the removed penultimate column (Fig. 1) was about 1.2 times that of the one-bay RC frame with an infilled wall in Specimen PCR [Fig. 2(a)]. The numerical models were built using the commercial structural software SAP2000 (CSI 2000), in which the beams with flange slabs and the columns were modeled by frame elements and the infilled wall was modeled by shell elements. The cross-sectional properties of the elements, i.e., the dimension, reinforcement, and material properties, were defined according to the design parameters of the specimens. The elastic story drift of the frame structure was limited to 1=550 of the story height, as required by the design codes (MOHURD 2010a, b). Hence the maximum acceptable horizontal displacement (1,200 mm/550 = 2.2 mm) was assigned to the frame joints to obtain the lateral stiffness. Therefore, the simplification of this specimen having a one-bay frame with an infilled wall was capable of providing a strong horizontal constraint, approximately equivalent to that provided by the original four-bay RC frame. Similarly, each of the original two-bay frames on either side of Specimen ICR was simplified by a one-bay RC frame with an infilled wall.

The cross-sectional dimensions and reinforcement details of the specimens are shown in Fig. 3. The reinforcement of the two specimens was identical and both had the same reinforcement ratio as that of the prototype structure. Given that the horizontal tensile forces in the floor slabs and frame beams may lead to the failure of the columns, the flange slabs with an effective width were also fabricated. Note that the effective width was regulated by the Chinese design code (MOHURD 2010a), which was six times the slab thickness. The concrete cover was 8 mm for beams and columns and 7 mm for slabs. C30
concrete was adopted for the specimens and the design strength $f_{cu}$ (i.e., a $150 \times 150 \times 150$ mm cubic compressive strength) was 29.2 MPa at 28 days. The material test data for reinforcement are given in Fig. 4, averaged from three tests for each type of steel bars.

**Test Setup and Instrumentation**

For PCR, two jacks were mounted on the top of both the removed penultimate and the edge columns. A load of 530 kN (taking the design axial compression ratio as 0.6) was applied on the edge column and kept unchanged throughout the test to simulate the gravity load from the upper stories. Subsequently, an increased displacement was applied on the top of the removed penultimate column head. For ICR, only an increased displacement was applied onto the removed inner column head. In either specimen, three linear variable differential transducers (LVDTs) were installed to monitor the vertical displacement of the removed column (D-1), the horizontal displacement of the left-hand RC wall (D-2), and the edge column (D-3), as shown in Fig. 2. The test data indicated that limited horizontal displacement was developed in the RC walls confirming that the horizontal constraint was reliably produced. Strain gauges at the key sections (for PCR: A-H; for ICR: A-F) of reinforcing bars in the beams (B1 to B4), the slabs (S1 to S8) and the edge column (C1 to C2) were installed as shown in Figs. 3(a-b). At the mid-span of the beams, i.e., Sections B and E (Fig. 2), four concrete strain gauges (BC-1 to BC-4) were installed at the mid-depth of the beams on both side surfaces to measure the variation of the axial forces developed within. At the cross section which is 12.5 mm away from the top and bottom of the edge column, i.e., Sections G and H, another four concrete strain gauges (CC-1 to CC-4) were placed on both inner and outer surfaces of the column, as shown in Fig. 2(a).
Experimental Results

Overall Structural Behaviour

The load-displacement curves of the two specimens are shown in Fig. 5, in which the applied load recorded at the removed column is plotted against the vertical displacement measured at the same location. Three critical points are marked on each curve: the peak point A in the stage of CAA, the peak point C in the stage of CA and the transition point B between the two stages. The coordinates of these points are defined as $(D_b, F_b)$, $(D_c, F_c)$ and $(D_t, F_t)$ and their specific values are displayed in Fig. 5.

For Specimen PCR, when the vertical displacement reached 5 mm, cracks occurred on the bottom surfaces of Sections C and D. The bending cracks began to develop on the top surfaces of Sections A and F and the bottom surfaces of Sections C and D when the displacement reached 25 mm. At 65 mm, concrete spalling was observed at the bottom of Section A, revealing that the flexural capacity at the beam end started to decline. When the displacement reached 70 mm, concrete spalling was also noticeable at the bottom of Section F. At 90 mm, horizontal cracks developed on the outer surface of Section G. The steel bars at the bottom of Sections F and A began to buckle at the displacements of 105 mm and 170 mm, respectively. At 190 mm, horizontal cracks started to develop on the outer surface of Section H. At 250 mm, concrete spalling occurred on the inner surface of Section H and the applied load started to decline. When the displacement reached 270 mm, two steel bars at the bottom of Section D ruptured successively and the applied load also dropped abruptly. Afterwards, the applied load increased only slightly followed by continued declining due to the increased damage of the edge column. Crushing of the concrete occurred on the inner surface of Section H and the steel bars in the edge column buckled at the displacement of
390 mm, leading to a complete failure of the column which marked the termination of the test. The ultimate failure mode is shown in Fig. 6(a).

For Specimen ICR, the overall load-displacement response and the failure phenomena were similar to the existing ICR experiments (Ren et al. 2016). When the displacement reached 365mm, two steel bars on the bottom surface of Section C ruptured successively which caused an abrupt load drop. Despite the rupture of steel bars, the load continued to rise with an increase in displacement. The ultimate displacement of this specimen was 520mm and the corresponding ultimate failure mode can be seen in Fig. 6(b).

**Progressive Collapse Mechanism under CAA**

The steel strains recorded at Sections A, D and E are presented in Figs. 7(a), (b) and (c and d), respectively, in which the tensile strains are in positive values while the compressive strains are negative. In the CAA stage, Section E in both specimens was close to the point of inflection and acted primarily in compression: under a limited tension on the top of the slab-flange beam (gauges B2, S1 to S6) and a significant compression at the bottom (Gauge B4), as shown in Figs. 7(c and d). Note that the compressive strain at the bottom of Section E in PCR (maximum compressive strain of 618 µε at B4) was around 50% smaller than that in ICR (maximum compressive strain of 1147 µε at B4). This indicates that the induced axial force in PCR was much smaller than that in ICR. Note that the collapse resistance was provided by the flexural capacity at the beam ends and the CAA. The fact that a smaller axial force in the CAA led to a smaller increase in the flexural capacity, the peak resistance of CAA of Specimen PCR (69 kN) was thus 9% smaller than that of Specimen ICR (76 kN) (see Fig. 5). Furthermore, the edge column in Specimen PCR translated 3 mm outward (corresponding to \( D_b \) in Fig. 8(a)) which caused a partial release of the axial deformation of the beams. Hence,
the displacement $D_b$ of PCR (65 mm) in the stage of CAA was 25% larger than that of ICR (52mm).

**Progressive Collapse Mechanism under CA**

At the CA stage, the steel strains, measured from all the strain gauges in the slab-flange beams in ICR, underwent a sustained and rapid growth (see Fig. 7(c)) due to the sufficient constraints on both ends of the specimen. Consequently, the collapse resistance under CA ($F_c$ in Fig. 5) also improved significantly and even exceeded the peak resistance of CAA ($F_b$ in Fig. 5). However, the strains of all the steel bars in the slab-flange beams in PCR developed slowly in the CA stage (Fig. 7(d)). As a result, the resistance at the large deformation stage did not increase notably ($F_c$ being slightly larger than $F_l$). The reason for this phenomenon is: for Specimen PCR, due to the limited constraint at one end of the beam, the top of the edge column moved inwardly reaching a maximum horizontal displacement of 58.5 mm corresponding to $D_c$ [Fig. 8(a)].

**Horizontal Constraints Provided by the Edge Column in Specimen PCR**

For PCR, the axial compressive force in its slab-flange beams pushed the edge column outward under the small deformation stage. When this axial force reached its peak, the horizontal displacement at the top of the edge column reached 3 mm accordingly [Point A in Fig. 8(a)]. After the peak, the CAA gradually decreased. Because the steel bars in the edge column did not yield at this stage, noting that the yielding strain $\varepsilon_y$ is about 3650 $\mu\varepsilon$ as shown in Fig. 8(b), this column had the capacity for deformation recovery and its tip started to move oppositely towards the frame. The horizontal displacement at the top of the column reached 3 mm inwardly at the transition point B [Fig. 8(a)]. After the transition, the column
was subjected to a tensile force transferred from the beams leading to a rapid yielding of steel bars [Fig. 8(b)] and crushing of the concrete [Fig. 8(c)] at the column base. When the edge column failed, the specimen became unstable although the slab-flange beams could provide further resistance.

**Comparison of Internal Forces Estimated by the Measured Strains**

The axial forces and bending moments at the key sections were obtained by integrating the stress distributions over the cross-sections. The compressive stress of concrete, \( \sigma_c \), can be calculated by Eq. (1) (Hognestad et al. 1955) as follows:

\[
\sigma_c = \begin{cases} 
  f_c \left( 2 \frac{\varepsilon_c}{\varepsilon_0} - \left( \frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right) & \varepsilon_c \leq \varepsilon_0 \\
  f_c \left( 1 - 0.15 \frac{\varepsilon_c - \varepsilon_0}{\varepsilon_u - \varepsilon_0} \right) & \varepsilon_0 \leq \varepsilon_c \leq \varepsilon_u 
\end{cases}
\]

where \( f_c \) = uniaxial compressive strength of concrete; \( \varepsilon_c, \varepsilon_0 \) and \( \varepsilon_u \) = strain, peak strain, and ultimate compressive strain of concrete, respectively, in which \( \varepsilon_0 = 0.002 \) and \( \varepsilon_u = 0.0038 \).

The uniaxial tensile stress of concrete, \( \sigma_{ct} \), was calculated by Eq. (2) (CEB-FIP 1990), noting that the contribution of the tensile force of cracked concrete is neglected in the equation, given as

\[
\sigma_{ct} = \begin{cases} 
  E_{ct} \varepsilon_{ct} & \sigma_{ct} \leq 0.9 f_{ctm} \\
  f_{ctm} \left( 0.00015 - 0.9 f_{ctm} / E_{ct} \right) (0.00015 - \varepsilon_{ct}) & 0.9 f_{ctm} \leq \sigma_{ct} \leq f_{ctm} 
\end{cases}
\]

where \( E_{ct} \) = tangent modulus of elasticity in (MPa), \( f_{ctm} \) = tensile strength in (MPa); and \( \varepsilon_{ct} = \) tensile strain of concrete.

The stresses of the steel bars were calculated according to Fig. 4. A schematic of the force equilibrium diagrams at the CAA and CA stages are shown in Fig. 9.
**CAA Stage**

Based on Fig. 9(a) for beam AC (similarly for beam DF), a mathematical equation of the load applied, $P_{CAA}$, onto the removed column can be expressed in Eq. (3) based on force equilibrium.

$$P_{CAA} = \frac{(M_A + M_C + M_D + M_F) + (N_{AC} \cdot d_{AC} + N_{DF} \cdot d_{DF})}{l_n}$$

(3)

where $M_A$ and $M_C$ = beam-end moments; $N_{AC}$ = axial compressive force in beam AC; $d_{AC}$ = relative vertical position of $N_{AC}$ at two ends of beam AC due to its deformed configuration; $l_n$ = clear span of the beam. $M_D$, $M_F$, $N_{DF}$ and $d_{DF}$ are defined similarly for beam DF relative to beam AC.

The underlying methodology for Eq. (3) was based on Park’s theory (Park and Gamble 2000), in which the bending capacity of the beam ends ($M_A$, $M_C$, $M_D$ and $M_F$) and the additional bending moments induced by the axial compressive forces ($N_{AC}$ and $N_{DF}$) are both taken into account. Due to severe plastic damage of Sections C and D in Specimens PCR and ICR, the strain gauges at these sections failed before reaching the peak of CAA, a phenomenon that can also be found in similar tests (Yi et al. 2008). For this reason, only the structural resistance in the CAA stage before damaging of the gauges at the beam ends was calculated.

Fig. 10(a) shows the comparison between the calculated (using Eq. (3)) and experimental resistance values, $P_{CAA}$ of PCR and ICR. A good agreement was demonstrated, except that at the early stage of the test, the calculated values of $P_{CAA}$ were lower than the experimental ones for both specimens. Note that the term $(N_{AC} \cdot d_{AC} + N_{DF} \cdot d_{DF})/l_n$ in Eq. (3) represents the resistance contributed by the axial compressive forces in the stage of CAA. The percentage
contribution of CAA to the overall resistance is presented in Fig. 10(b). In the early stage of the test, due to the smaller horizontal constraint in PCR, its CAA contribution increased slower than that in ICR and its peak contribution was 16% lower than that in ICR. For both specimens, it is worthwhile noting that the vertical displacements at the removed columns corresponding to the peak CAA contributions ($D_p=6\text{mm}$ for ICR and $D_p=9.5\text{mm}$ for PCR in Fig. 10(b)) were different from those corresponding to the peak resistance under the CAA stage ($D_b=52\text{mm}$ of ICR and $D_b=65\text{mm}$ of PCR in Fig. 5). This was because the contribution of CAA was determined by the additional bending moment induced by the axial compressive forces in the slab-flange beams, i.e., $N_{AC}\cdot d_{AC}+N_{DF}\cdot d_{DF}$. Although the magnitude of the axial forces ($N_{AC}$ and $N_{DF}$) continued to increase during the loading stage, the relative vertical positions of $N_{AC}$ and $N_{DF}$ at beam ends (i.e., $d_{AC}$ and $d_{DF}$) increased initially due to an anti-clockwise rotation of the neutral axis, but subsequently decreased due to the significant increase of the vertical displacement of the removed column. At the ultimate displacement, the calculated contribution of CAA in PCR was 21% lower than that in ICR. This confirmed that the CAA was unable to be fully developed when the horizontal constraint is insufficient, as in the case of PCR.

### CA Stage

As the force equilibrium diagram shown in Fig. 9(b), a mathematical equation of the load applied onto the removed column in the stage of CA, $P_{CA}$, can be expressed in Eq. (4) as follows:

$$P_{CA} = (T_B + T_E) \cdot \sin \theta$$

where

$$\sin \theta = \frac{\delta}{\sqrt{\delta^2 + (l_n - 2l_p)^2}}$$
where \( l_p \) = length of the plastic hinge, assumed as 0.25m; \( \delta \) = vertical displacement of the removed column; and \( T_B \) and \( T_E \) = axial tensile forces in the steel bars measured in Sections B and E, respectively.

Under the CA stage (from the transition Point B to the rupture of steel bars), the calculated and experimental resistance values \( P_{CA} \) of the two specimens PCR and ICR are compared in Fig. 11. In both specimens, the calculated \( P_{CA} \) values, provided by the steel bars at Sections B and E were lower than the experimental ones, which indicates that not only the axial forces in the slab-flange beams but also the residual flexural moments at plastic hinges at the beam ends contributed to the overall resistance. Note that there were considerable differences between the two specimens. For ICR, the calculated \( P_{CA} \) values contributed by the steel bars at Sections B and E continued to approach the experimental resistance values. Finally, the ultimate resistance was completely provided by the CA of the steel bars. This was because the residual flexural capacity at the beam ends continued to decrease due to the increasingly intensified rotational deformation at the same ends. For PCR, however, the contribution of the axial forces provided by the steel bars, calculated at the end of the CA stage, was only about 35% of the experimental resistance. This was because partial axial and rotational deformations of the slab-flange beams were released due to the horizontal movement and bending rotation of the edge column. The release of the axial deformation slowed down the development of the axial forces in the slab-flange beams, but the release of the rotational deformation did not affect the residual flexural capacity. Hence, the ultimate resistance at the final stage was provided not only by the CA of the steel bars but also by the residual capacity of the beam ends.
Vulnerability of the Prototype RC Frame

The current progressive collapse design methods for frame structures are primarily focused on the bearing and deformation capacities of beams and slabs, whereas considerations of the bearing capacity of columns which provide horizontal constraints have been fairly limited. Generally, the bearing capacities of columns are largely governed by gravity and seismic designs, which may not meet the demand of progressive collapse design. As a result, the remaining columns surrounding the initially failed column may be further damaged in a progressive collapse scenario, which may in turn lead to the damage of the horizontal constraint and the reduction of the progressive collapse resistance. In order to assess the progressive collapse vulnerability of RC frame structures designed by the existing code methods, this section analyses the bending moments of the columns under different design conditions. A single planar frame of the prototype structure shown in Fig. 1 is presented in Fig. 12. Subsequent to the failure of a penultimate column on the ground floor (Case 1) and on the top floor (Case 2), the adequacy of the horizontal constraint that the adjacent columns could provide was also studied. Note that the edge beams in the Y-direction (Fig. 1) were designed to bear the gravity load only, in which the reinforcing bars were placed at the top and bottom of the beam sections, therefore providing limited horizontal constraint to the selected planar frame.

According to the Chinese codes (MOHURD 2010a, 2010b), the bending moments of the edge column and inner column designed for gravity and seismic actions were calculated. According to the Unified Facilities Criteria (DoD 2016), the bending moment demands for progressive collapse-resistant design were also determined. The calculated bending moments in the columns are provided in Table 1, where EC and IC represent the edge and
inner columns, respectively; B and T represent the bottom and top of the column, respectively. For the design seismic intensities of 6, 7, 8 and 9 degrees, the corresponding peak ground accelerations are 0.05g, 0.1g, 0.2g and 0.4g, respectively, for the design earthquake (with a 10% probability of exceedance in 50 years) where g is the acceleration of gravity. It can be found that these bending moments were dominated by the conventional seismic design.

**Case 1**

For the prototype frame, the design moment capacities of the bottom and top of the edge column on the ground floor under the seismic intensity of 8 degrees were 2151 kN∙m and 701 kN-m, respectively. These values were much larger than the demands of progressive collapse design under small deformations (i.e., 133 kN∙m and 252 kN-m, respectively), when a penultimate column on the same floor was damaged (Case 1 in Fig. 12). This suggests that the seismic design can meet the demand of progressive collapse design under small deformations, which was consistent with the experimental results that ICR remained stable in the stage of CAA. Under large deformations, on the other hand, the progressive collapse design demand for the column base (i.e., 30869 kN-m) was about 14.4 times more than the seismic design value (2151 kN-m). This indicates that the seismic design is far from being able to meet the progressive collapse demand for a large deformation behaviour. This also confirms the experimental finding presented earlier that the edge column failed before the full development of CA of the slab-flange beams in PCR. In the Unified Facilities Criteria (DoD 2016), the alternative load path (ALP) method and the tie force (TF) method are used to examine the CAA and CA capacities of beams and slabs, respectively. Unfortunately, the horizontal constraints to the beams and slabs are not required to be checked in the TF
method. The calculated bending moments in the columns presented herein reveals that when an RC frame structure subjected to a penultimate column removal scenario, the ALP method can offer safe capacity predictions whereas the prediction due to the TF method would be unsafe due to the subsequent failure of the edge column.

Seismic design has been considered to inherently enhance the progressive collapse resistance of building structures (Li et al. 2011). When the ground floor undergoes small deformations, the design seismic capacities of the edge and inner columns can meet the progressive collapse demand when the intensity was higher (7, 8 or 9 degrees). When the seismic intensity was decreased to 6 degrees, the progressive collapse resistance demands of the tops of the edge and inner columns were 1.15 and 1.34 times as much as the corresponding design seismic capacities, respectively, hence not meeting the progressive collapse demand. This indicates that for RC frame structures with lower seismic requirements, the columns are required to be further checked by using the ALP method to ensure that sufficient horizontal constraints are provided, even for the CAA capacity design. On the other hand, under large deformations, the progressive collapse resistance demands of the edge column were about 7.2 (9 degrees), 14.4 (8 degrees), 28.3 (7 degrees) and 54.9 (6 degrees) times more than the design seismic capacities, which was far from being able to meet the demand of progressive collapse design. The demands of the inner column were about 1.3 (9 degrees), 2.6 (8 degrees), 5.2 (7 degrees) and 10.4 (6 degrees) times as much as the design seismic capacities, which was also unable to meet the progressive collapse resistance requirement. Hence, the bearing capacity of the columns in the periphery area of the building should be increased if the CA capacity of the structure is designed by the TF method.
Case 2

When a penultimate column on the top floor fails (Case 2 in Fig. 12), neither the bending moment at the top of edge column nor the inner column was able to meet the requirement of progressive collapse design under small deformations, which would cause the failure of the column top. Hence, compared with the ground floor, the progressive collapse vulnerability of the top floor was higher. As such, the frame columns in the periphery area of the top floor should also be enhanced to prevent potential horizontal propagation of a progressive collapse.

Conclusion

1. The experimental results indicated that under the CAA, the peak resistance of Specimen PCR having insufficient horizontal constraints was 9% smaller than that of Specimen ICR. The displacement corresponding to the peak resistance in PCR was 25% larger than that in ICR. Under the CA, the collapse resistance of ICR improved significantly and even exceeded the peak resistance of CAA. However, the resistance of PCR did not increase notably.

2. The internal force calculation demonstrates that the contribution of CAA in PCR was 21% lower than that in ICR. The CA in PCR developed fully, and the ultimate resistance was solely provided by the tensile forces in the steel bars. For ICR, the axial and rotational deformations of the slab-flange beams were partially released. Therefore, the contribution of the axial forces provided by the steel bars was limited (only 35%).

3. For the prototype frame, in the case that a penultimate column on the ground floor was damaged, the seismic design for the edge and inner columns on the same floor can meet the progressive collapse design requirement under small deformations, when the design seismic
intensities were 7 degrees and higher. However, the design seismic of the column tops cannot satisfy the progressive collapse demand when the seismic intensity was lower (6 degrees). Under large deformations, the progressive collapse demand of the edge and inner columns were about 7.2 (9 degrees) to 54.9 (6 degrees) and 1.3 (9 degrees) to 10.4 (6 degrees) times the corresponding seismic design values, respectively, which was far from being able to meet the demand of progressive collapse design.

Also for the prototype frame, when a penultimate column on the top floor was damaged, neither the bending moment of the top of the edge column nor the inner column was able to meet the demand of progressive collapse design under small deformations. Compared to the ground floor, the progressive collapse vulnerability of the top floor was higher.

Data Availability Statement

All data, models, and code generated or used during the study appear in the submitted article.

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References


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Fig. 1. Prototype structure (dimensions in m).

Fig. 2. Detailed dimensions of specimens (dimensions in mm): (a) Specimen PCR; (b) Specimen ICR.
Fig. 3. Cross-sectional dimensions and reinforcement details of test specimens (dimensions in mm): (a) Slab-flange beam; (b) Edge column; (c) Sub-beam; (d) RC wall; (e) Footing beam.

Fig. 4. Measured stress-strain curves of steel bars.
Fig. 5. Load-displacement curves.

Fig. 6. The ultimate failure modes of specimens: (a) Specimen PCR; (b) Specimen ICR.
Fig. 7. Development of steel strain at typical sections in beams: (a) Section A; (b) Section D; (c) Section E of ICR; (d) Section E of PCR.
Fig. 8. Horizontal displacement at column tip and strain development in PCR: (a) Horizontal displacement; (b) Strain in steel bars; (c) Strain in concrete.

Fig. 9. Schematic of force equilibrium diagrams for: (a) CAA stage for beam AC; (b) CA stage for beams AC and DF.
**Fig. 10.** Resistance $P_{CA}$ in the CAA stage and contribution of CAA: (a) Calculated and experimental resistance results; (b) Contribution of CAA.

**Fig. 11.** Calculation of resistances under CA stage.

**Fig. 12.** Penultimate column failure cases.