A novel structural detailing for the improvement of seismic and progressive collapse performances of RC frames

A NOVEL STRUCTURAL DETAILING FOR THE IMPROVEMENT OF SEISMIC AND PROGRESSIVE COLLAPSE PERFORMANCES OF RC FRAMES

Kaiqi Lin[1,2], Xinzheng Lu[3,*], Yi Li[4], Weidong Zhuo[1], and Hong Guan[5]

[1] College of Civil Engineering, Fuzhou University, Fuzhou, China

[2] Department of Civil and Environmental Engineering, The Hong Kong Polytechnic University, Hong Kong, China

[3] Key Laboratory of Civil Engineering Safety and Durability of China Education Ministry, Department of Civil Engineering, Tsinghua University, Beijing, China

[4] Key Laboratory of Urban Security and Disaster Engineering of Ministry of Education, Beijing University of Technology, Beijing, China

[5]School of Engineering and Built Environment, Griffith University Gold Coast Campus, Queensland, Australia

Correspondence
Xinzheng Lu, Key Laboratory of Civil Engineering Safety and Durability of China Education Ministry, Department of Civil Engineering, Tsinghua University, Beijing, P.R. China, 100084.
E-mail: luxz@tsinghua.edu.cn

Earthquake-induced building collapse and progressive collapse due to accidental local failure of vertical components are the two most common failure modes of reinforced concrete (RC) frame structures. Conventional design methods usually focus on the design requirements of a specific hazard but neglect the interactions between different designs. For example, the progressive collapse design of an RC frame often yields increased reinforcement and flexural strength of the beams. As a result, the seismic design principle of "strong-column-weak-beam" may be violated, which may lead to unfavorable failure modes and weaken the seismic performance. To avoid these adverse effects of the progressive collapse design on the seismic resistance of RC frames, a novel structural detailing is proposed in this study. The proposed detailing technique intends to concurrently improve the seismic and progressive collapse performances of an RC frame by changing the layout of the newly added longitudinal reinforcement against progressive collapse without introducing any additional reinforcement. A six-story RC frame is used as the prototype building for this investigation. Both cyclic and progressive collapse tests are conducted to validate the performance of the proposed structural detailing. Based on the experimental results, detailed finite element (FE) models of the RC frame with different reinforcement layouts are established. The seismic and progressive collapse resistances of different models are compared based on the incremental dynamic analysis (IDA) and nonlinear dynamic alternate path (AP) methods, respectively. The results indicate that the proposed structural detailing can effectively resolve the conflict between the seismic and progressive collapse designs.

KEYWORDS

RC frame, Structural detailing, Seismic resistance, Progressive collapse, Strong-column-weak-beam failure

INTRODUCTION

In recent years, research on the multi-hazard resistances of building structures has attracted increasing attention in civil engineering community. A reinforced concrete (RC) frame is one of the most commonly used structural systems. According to existing studies, earthquake-induced building collapse and progressive collapse due to accidental local failures are the two most commonly encountered hazards that have adverse impacts on the safety of multistory RC frames.[1,2] Consequently, many design codes or guidelines have been issued for the seismic or progressive collapse designs of RC frames.[3-6] Furthermore, the seismic and progressive collapse designs are both mandatory in some countries or regions for buildings of high level of importance.[3,4]

As a matter of fact, the seismic design of RC frames differs significantly from the progressive collapse design. The seismic design intends to resist a global lateral inertial force caused by ground motion. The extensively used

capacity design concept requires that the strengths of the beams and adjoining columns should adhere to the "strong-column-weak-beam" principle.[3,7] Hence, the seismic resistance of frame columns is a key factor of the seismic performance of an RC frame.[2]

In comparison with the earthquake-induced collapse, progressive collapse refers to the disproportionate collapse behavior of a structure initiated by a small and localized failure which may be caused by fire, explosion, or overloading.[8] Prevention of progressive collapse includes reducing the probability of initial failure (which is not always feasible) and improving the resistance of the structure.[9] The progressive collapse resistance of an RC frame is considerably dependent on the load redistribution and bridging capacities of the horizontal components above the failed region. Consequently, the strengths of RC frame beams are a major concern for most existing progressive collapse design regulations.[5,10,11] Amongst the existing studies, the alternate path (AP) method is most commonly used for design and evaluation of the structural progressive collapse resistance.[12-17] In the AP method, the structure is examined under the removal of one or several critical columns that are vulnerable to hazards. This method is a threat-independent method, making it universal for the progressive collapse design against different hazards. The method is also recommended by many current design guidelines and/or specifications to study the progressive collapse behaviour of various structures.[5,10,11] Hence, the AP method is also adopted herein and the most widely used middle-column removal scenario[12-17] is considered in the proposed experimental tests.

According to many published studies, when the seismic design and progressive collapse design are considered separately, the design requirements can be easily met according to the corresponding design codes.[3,5,7,10,11] However, owing to the different design principles of seismic and progressive collapse designs, when both design requirements need to be fulfilled simultaneously, existing design methods may not yield ideal results. Li et al[18] pointed out that mitigating risks against an individual hazard would affect the structural vulnerability to other hazards from the perspective of the structural design. Lin et al[19] found that an RC frame with relatively low-seismic-design intensity can hardly meet the requirements of the progressive collapse design, and its beams should be strengthened to prevent progressive collapse. However, after performing the progressive collapse design, the flexural capacities of the frame beams will be increased owing to the newly added reinforcement, thus, leading to an unfavorable "strong-beam-weak-column" failure mode under seismic loads, which will, in turn, weaken the seismic performance of the structure. Feng et al[20] also concluded that the RC frame may experience a "strong-beam-weak-column" failure through the pushover analysis of a six-story, two-bay RC frame, after the implementation of a current progressive collapse design. Hence, it is necessary to develop an effective engineering solution to resolve the conflict between the seismic and progressive collapse designs and to improve the multi-hazard resistance of RC frames.

In view of the aforementioned demands, a novel structural detailing is proposed in this study. By changing the layout of the newly added reinforcement in the RC beams against progressive collapse, the proposed structural detailing can optimize the failure mode of an RC frame under seismic loads, while providing sufficient progressive collapse resistance. In the published literature, many detailing techniques have been developed for improving the seismic or progressive collapse performances of RC frames, taken into consideration the significant values of a rational structural detailing to structural safety. Eom et al[21] summarized different detailing techniques for the beam-column joints by relocating the beam plastic hinges away from the beam-column interface. The seismic performance of two strengthening methods using 90° hooked and headed bars, and a weakening method using reduced section bars, were experimentally compared with the conventional beam-column joints. Qu et al[22] proposed an unconstrained gusset connection for the buckling restrained braces in RC frames to mitigate the complicated and harmful interactions between the gusset plate and the adjoining columns. Sasmal and Nath[23] proposed a diagonal metallic single haunch/bracing system for protecting the beam-column joints of poorly designed structures under seismic cyclic loading. In addition to these research efforts, other researchers have investigated the effects of different types of structural detailing on the progressive collapse resistance of RC beams. Yu and Tan[24] compared four different detailing techniques for improving the progressive collapse resistance of an RC beam and provided some useful suggestions. Alogla et al[25] experimentally compared the progressive collapse resistances of RC beams with different rebar locations. Their results indicated that the additional rebars near the bottom of the beam could improve the progressive collapse resistance more efficiently. Li and Sasani[26] and Livingston et al[27] evaluated the effects of structural integrity requirements on the progressive collapse resistance of RC frames. However, the aforementioned studies of structural detailing mainly focused on the improvement of the structural resistance against a specific hazard. The identification of ways to ensure the seismic performance of an RC frame after implementing the progressive collapse design is still an unsolved problem from the perspective of multi-hazard mitigation of building structures.

To validate the efficacy of the proposed structural detailing on the seismic and progressive collapse resistances of RC frames, a six-story RC frame designed according to the Chinese design codes is chosen as the prototype building. Both cyclic and progressive collapse tests are conducted to evaluate the multi-hazard resistances of (1) a conventional RC frame (i.e., RC6), (2) the RC frame after the implementation of the conventional progressive collapse design (RD1), and (3) the RC frame incorporating the newly proposed structural detailing (ND). Based on the experimental results, a modeling strategy specifically for the detailing is proposed to establish the finite element (FE) model of the RC frame with the newly proposed structural detailing. Finally, the effectiveness of the proposed detailing technique in improving the seismic and progressive collapse performances is further validated based on the incremental dynamic analysis (IDA) and nonlinear dynamic AP analysis of different RC frame models.

EXPERIMENTAL SCHEME

Design of the prototype RC frame

The elevation and plan views of the prototype RC frame are shown in Figure 1. The first story is 4.2 m in height, while the remaining stories are 3.6 m in height. The span lengths in the longitudinal and translational directions are 6 m. The first story columns are considered to be fully fixed to the ground. The dead load considered on each story is 5.0 kN/m^2, whereas the live load is 2.0 kN/m^2. More detailed information about this prototype frame can be found in Lin et al[17,19] and Ren et al.[16] The structure is firstly designed according to the Chinese design codes (i.e., the Code for the Design of Concrete Structures[6] and the Code for the Seismic Design of Buildings[7]) to create the conventional RC frame, i.e., RC6. The seismic design intensity is VI, corresponding to a design peak ground acceleration (PGA) with a 10% probability of exceedance in 50 years equaling 0.05g.

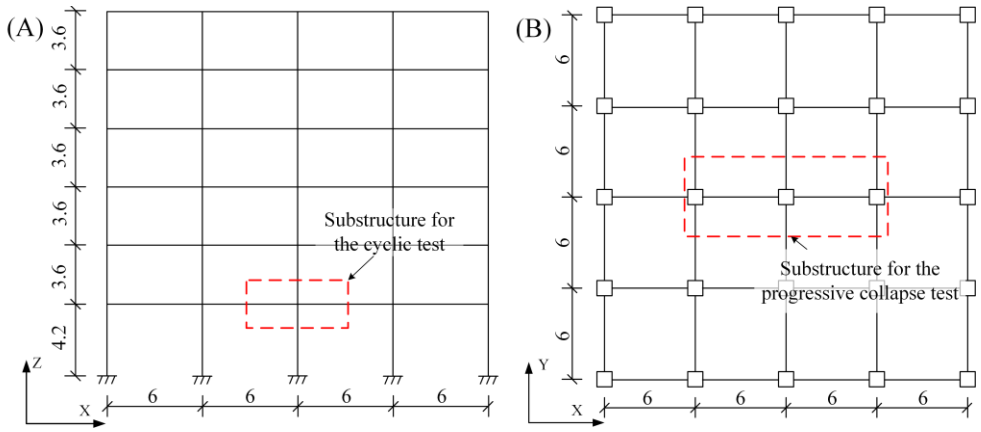

Figure 1 Elevation view (A) and plan view (B) of the six-story RC frame (unit: m).

According to the numerical results of Lin et al,[19] progressive collapse occurred in the prototype building when any column on the first to fifth story was removed. This indicated that RC6 could not meet the code requirements of the progressive collapse design.[5,10,11] Therefore, RC6 is redesigned according to the tie force method provided in the DoD guideline,[10] and the new structure is named as RD1. Detailed design procedures for RD1 can be referred to Lin et al.[17]

The proposed reinforcement detailing

Overview of detailing

A comparison between the conventional reinforcement layout and proposed structural detailing is shown in Figure 2. The layout of the newly added anti-progressive collapse reinforcement is rearranged to improve the multi-hazard resistance.

(A) Conventional progressive collapse design

Conventionally, after performing the progressive collapse design, the newly added reinforcement for collapse prevention is usually placed at the top and bottom of the frame beam, as shown in Figure 2A. According to the experimental results presented in Lin et al[17], the flexural strength of the beam will increase by approximately 30% for RC6 under such a reinforcement layout. Hence, this layout will undoubtedly increase the flexural capacity of the beam, thus, resulting in a potential risk of "strong-beam-weak-column" failure under seismic actions.

(B) The proposed detailing technique

To solve the abovementioned problem, the key challenge is to maintain the progressive collapse resistance of the beam without overly increase its flexural strength. Hence, the proposed detailing technique re-arranges the newly added anti-progressive collapse reinforcement at the mid-height of the beam away from the column's face. In the vicinity of the joint interface, the added reinforcing bars are bent upward and downward at 45° towards the top and bottom surfaces of the beam, respectively, joining the original reinforcement to pass through the joint region, as shown in Figure 2B. Compared with the conventional reinforcement layout, the main advantages of the proposed detailing include:

(1) Around the reinforcement bent-up/bent-down region, the gradually changed height of the reinforcement will alter the flexural capacity of the beam along its length. Under an earthquake strike, it is possible to relocate the plastic hinge zone to the sections that have reduced flexural capacities by properly designing the sectional strength at the reinforcement bent-up/bent-down region. In doing so, the beam-column joint and the adjoining frame columns can be protected.

(2) Based on a column removal scenario, Li et al[28] revealed that the progressive collapse resistance of an RC frame is largely dependent on the catenary mechanism of the frame beam. Further studies by Yu and Tan,[14] Qian et al,[15] Ren et al,[16] and Lu et al,[29] and indicated that the resistance of an RC beam under the catenary mechanism is dominated by the area of the continuously arranged longitudinal reinforcement. Note that the proposed structural detailing does not change the total area of the continuously arranged longitudinal reinforcement in a beam. Consequently, the specimen with the new detailing can provide a similar progressive collapse resistance to the conventional one, as shown in Figure 2A.

As described above, an RC frame with the proposed detailing (Figure 2B) is expected to exhibit a better multi-hazard performance as compared to the conventional design shown in Figure 2A.

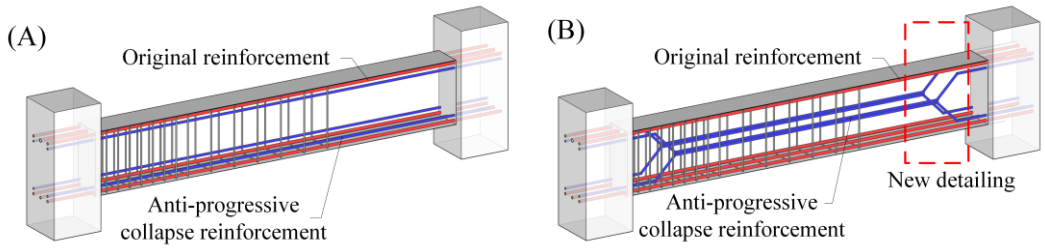

Figure 2 (A) Frame beam after progressive collapse design; and (B) Frame beam with new detailing.

Design requirements for detailing

The design scheme of the proposed detailing technique is depicted in Figure 3. The designed flexural strengths of the beams at the beam-column interface are M_{nj} and M_{nj}', where M_{nj} is the negative flexural capacity, and M_{nj}' is the positive one. The flexural strengths at the plastic hinge location are M_n and M_n', where M_n is the negative flexural capacity, and M_n' is the positive one. M_{uj} and M_{uj}' are, respectively, the negative and positive flexural demands at the beam-column interface corresponding to the flexural strengths at the relocated plastic hinge due to the concentrated load. Note that only the negative quantities are identified in the figure. The sectional details of the frame beam are also provided in Figure 3 in which A_{t1}, A_{t2}, A_{b1}, and A_{b2} are the areas of the reinforcement at the corresponding height of the beam. The red solid circles represent the original seismic reinforcement in the beam, while the blue hollow

ones denote the newly added anti-progressive collapse reinforcements. It is assumed that $A_{t1} \geq A_{b1}$ in the hogging moment region of the beam, and $A_{t2} = A_{b2}$ for the anti-progressive collapse reinforcement.

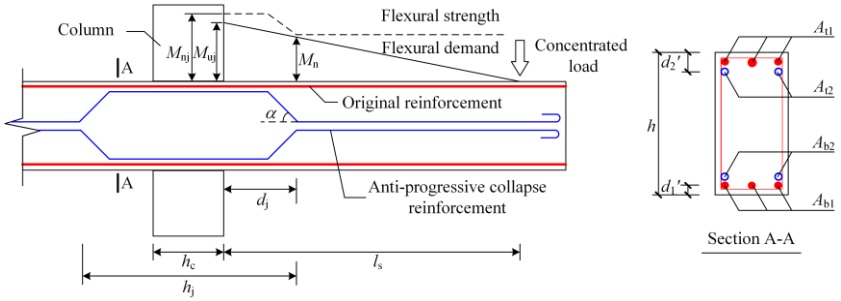

Figure 3 Design requirements for the detailing.

To relocate the plastic hinge zone, it is required that the flexural strengths M_{nj} and M_{nj}' are respectively larger than the flexural demands M_{uj} and M_{uj}' at the column face, as shown in Figure 3.[21]

$$M_{nj} \geq M_{uj} \text{ and } M_{nj}' \geq M_{uj}' \tag{1}$$

In the seismic cyclic tests in this study, the load is applied at the beam end. Hence, the flexural demands M_{uj} and M_{uj}' can be expressed by M_n and M_n', as shown in Equation 2:

$$M_{uj} = M_n l_s / (l_s - d_j) \text{ and } M_{uj}' = M_n' l_s / (l_s - d_j) \tag{2}$$

where M_n, M_n', M_{nj}, and M_{nj}' can be directly calculated according to the reinforcement areas at the corresponding sections, l_s is the distance between the loading point and the beam-column interface, and d_j is the distance between the relocated plastic hinge and the beam-column interface.

In addition, to prevent the frame from "strong-beam-weak-column" failure, the flexural strength of the column M_c should also comply with the requirements of "strong-column-weak-beam", as given in Equation 3:[3,7]

$$M_c \geq 1.2 M_{uj} \text{ and } M_c \geq 1.2 M_{uj}' \tag{3}$$

Furthermore, according to Eom et al,[21] the distance between the relocated plastic hinge and the beam-column interface must comply with Equation 4 below,

$$h_j / d_b \geq 20 \tag{4}$$

where d_b is the diameter of the rebars and h_j is the effective embedment length, as defined in Figure 3. Based on Equations 1 to 4, the tolerance for d_j can be derived as follows:

$$d_j \leq \min\left\{\left(1 - \frac{M_n}{M_{nj}}\right) l_s, \quad \left(1 - \frac{M_n'}{M_{nj}'}\right) l_s, \quad \left(1 - \frac{1.2M_n}{M_c}\right) l_s \quad \left(1 - \frac{1.2M_n'}{M_c}\right) l_s\right\} \tag{5}$$

$$d_j \geq (20 d_b - h_c)/2 \tag{6}$$

where h_c is the column width as shown in Figure 3. It is worth noting that when a proper d_j is determined based on Equations 5 and 6, any bending angle of the reinforcement is acceptable theoretically when the newly added anti-progressive collapse reinforcement can be bent to the mid-height of the beam within d_j. However, as the bending angle (i.e., α) increases, the flexural strength will change more abruptly within the reinforcement bent-up/bent-down

region, making it easier to form the plastic hinge at the bending position. Therefore, a bending angle equal to or greater than 45 degrees is recommended. Note that a 45-degree bending angle is widely used in the current construction practice. Consequently, such an angle was adopted in this work, and the objective of the proposed detailing was successfully satisfied. Nevertheless, further studies are needed to determine an appropriate bending angle, taking into the considerations of the material hardening effects after bending, ease of construction and so forth.

Experimental setup and specimens

To compare the seismic and progressive collapse performances of the conventional RC frame (RC6), the RC frame after the implementation of the progressive collapse design (RD1), and the RC frame with the newly proposed structural detailing (ND), two substructures enclosed by the red-dashed lines in Figures 1A and 1B are extracted from the building, respectively, for the seismic cyclic and progressive collapse tests. A more detailed calculation procedure for the progressive collapse design can be referred to Lin et al[17]. For the seismic cyclic tests, three frame joint specimens designated as S-RC6, S-RD1, and S-ND are examined. For the progressive collapse tests, three specimens designated as P-RC6, P-RD1, and P-ND are investigated. A commonly used 1/2-scale ratio reported in many published literature,[14,25,30-32] is adopted for all the specimens. It should be noted that the experimental results of the control specimens S-RC6, S-RD1, P-RC6 and P-RD1 have been presented in Lin et al. [17] As this study is focused on the performance of the specimens (i.e., S-ND and P-ND) with the newly proposed detailing, detailed and relevant experimental observations of these specimens can be referred to Lin et al.[17]

Seismic cyclic tests

The experimental setup for the seismic cyclic tests is shown in Figure 4A.[17] The joint specimens are hinged at both ends of the column. The distance between the top and bottom hinges is 1.9 m. During these tests, a constant vertical force of 486 kN corresponding to the design axial force ratio of 0.85 is firstly applied to the top of the column to simulate the load transferred from the upper stories. Subsequently, the seismic forces are simulated by gradually increasing the cyclic loads at the beam ends. The loading points on the beams are 1.5 m away from the joint center. Detailed loading protocol can be found in Lin et al.[17]

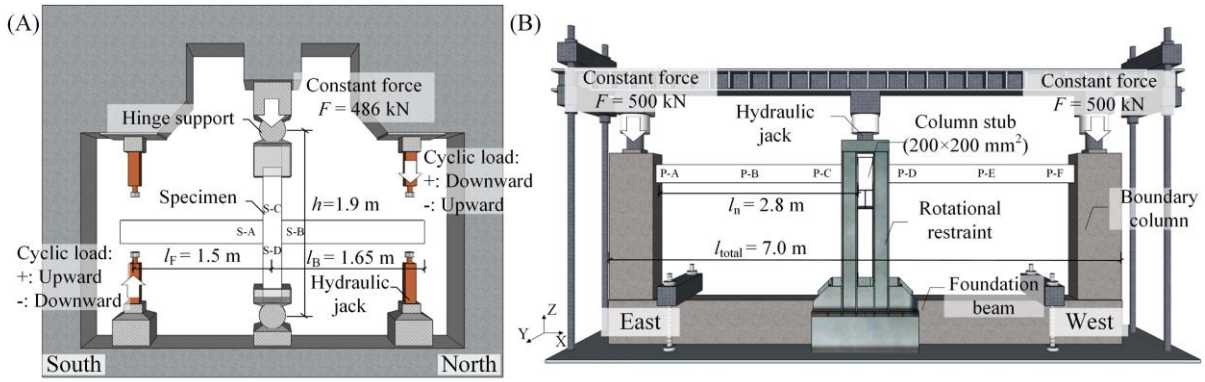

Figure 4 (A) Setup for the seismic cyclic test; and (B) Setup for the progressive collapse test.[17]

The relative rotation between the beam and column is calculated by measuring the displacements at the beam ends. The total moments (i.e., M) and the joint rotations (i.e., θ) of the specimens are calculated following Equations 7 and 8, respectively:

$$M = F_S \times l_F + F_N \times l_F \tag{7}$$

$$\theta = (\delta_S + \delta_N)/2l_F \tag{8}$$

where F_S and F_N are the forces recorded at the south and north loading points, respectively, δ_S and δ_N are the corresponding displacements, and l_F is the distance between the loading point and the joint center, as shown in

Figure 4A. For ease of description of the experimental results, four typical sections (Sections S-A to S-D) are defined on the joint specimen, as shown in Figure 4A.

The design drawings of Specimens S-RC6 and S-RD1 are depicted in Lin et al.[17] Based on Specimen S-RC6, two pairs of 8 mm diameter bars are respectively added at the top and bottom in the frame beams of Specimen S-RD1, as a result of the progressive collapse design (Figure 2A). Further, the reinforcement details of Specimen S-ND are shown in Figure 5, in which the blue dashed lines are used to represent the newly added anti-progressive collapse reinforcement. According to the plastic hinge relocation requirements and the actual reinforcement configurations (Figure 5), the flexural strengths at various critical sections of the beam can be calculated. Substituting the calculated flexural strengths into Equation 2 yields the hogging and sagging moments, as expressed in Equations 9-10.

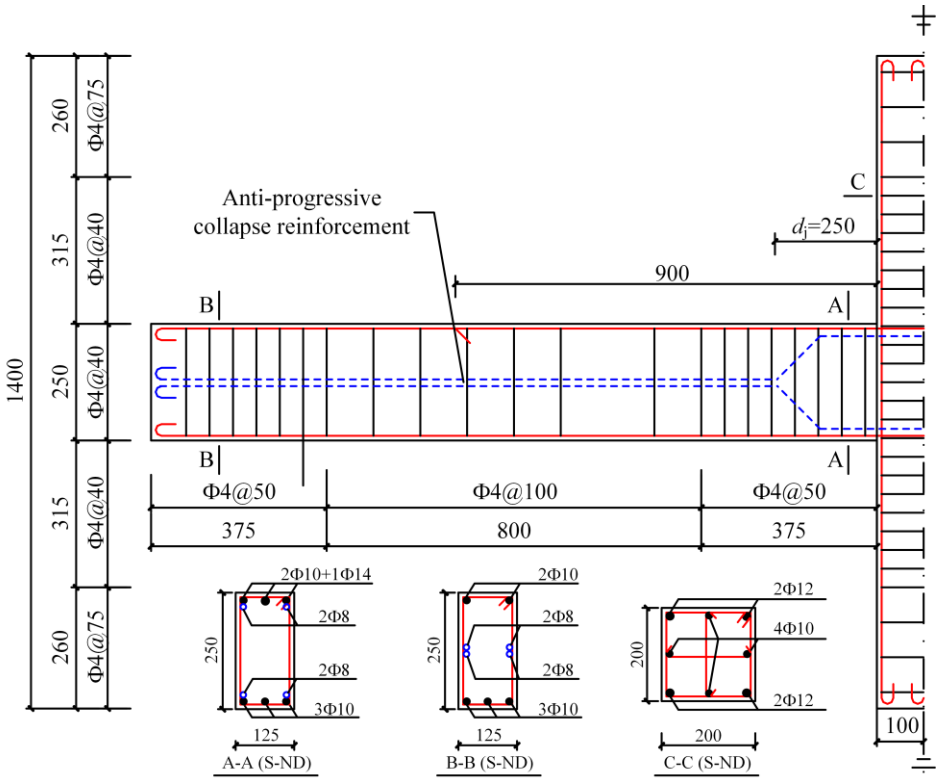

Figure 5 Reinforcement details of Specimen S-ND (unit: mm).

Hogging moment:

$$M_{nj} / M_n = 1.263 \geq l_s / (l_s - d_j) \tag{9}$$

Sagging moment:

$$M'_{nj} / M'_n = 1.326 \geq l_s / (l_s - d_j) \tag{10}$$

Based on Equations 9 and 10, it can be derived that $d_j \leq 0.291$ m. In addition, the flexural capacity of the column under the axial force of 486 kN is calculated as M_c = 40.70 kN·m. When the requirement of "strong-column-weak-beam" (Equation 3) is satisfied, it can be calculated that $d_j \leq 0.444$ m.

In summary, d_j should be equal to or less than 0.291 m based on Equation 5. Given that the anti-progressive collapse reinforcement is bent upward or downward at an angle of 45°, d_j is set to 0.25 m for the ND specimen to meet the requirements of Equations 5 and 6.

Progressive collapse tests

The two-span substructure on the ground floor of the prototype RC frame enveloped by the red-dashed rectangle in Figure 1B is used for the progressive collapse tests. The corresponding experimental setup is shown in Figure 4B. The two-span substructure is fixed by the strong boundary columns with widths of 600 mm, connecting to the foundation beam. To ensure the boundary fixities, H-shaped steel is embedded in the boundary column, and two constant vertical forces with a magnitude of 500 kN are applied to the top of the boundary columns.[17] The tests were conducted following the AP method, as specified in the progressive collapse design guidelines.[5,10,11] A monotonic concentrated load is applied to the stub of the removed column to simulate the progressive collapse of the substructure subjected to a middle-column removal scenario. The loading process is displacement-controlled. To prevent the rotation of the column stub along the X and Z axes (Figure 4B), rotational restraints are also installed in the place of the removed column. Six typical sections (Sections P-A to P-F) are defined on the specimens to better describe the experimental results, as shown in Figure 4B. Linear variable differential transducers (LVDTs) were installed at the four corners of the column stub and at the mid-span of each specimen (Section P-B and P-E) to monitor the deformation of the specimen.

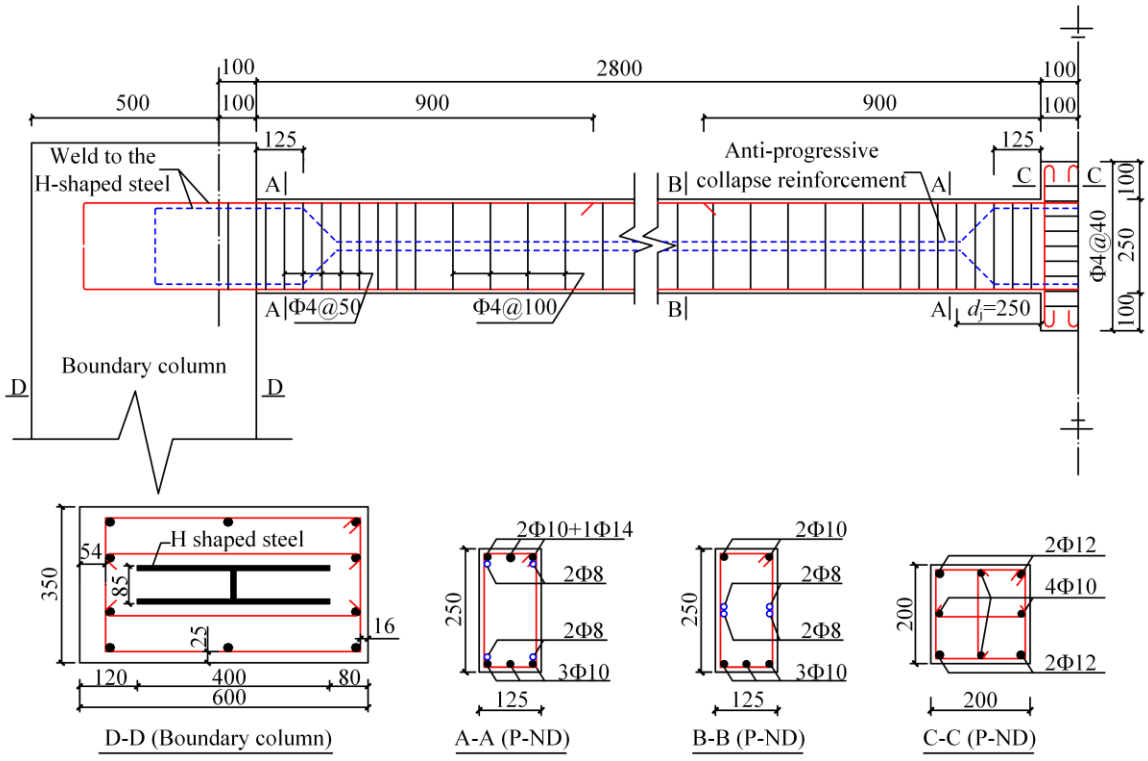

Figure 6 Reinforcement details of Specimen P-ND (unit: mm).

The reinforcement details of Specimens P-RC6 and P-RD1 can be referred to Lin et al.[17] Detailed drawings of Specimen P-ND are shown in Figure 6. The blue-dashed lines in Figure 6 also represent the newly added anti-progressive collapse reinforcement arranged following the proposed structural detailing. The longitudinal reinforcements are welded to the embedded H-shaped steel in the boundary columns thereby providing adequate anchorage strength.

Material properties

The material properties of the specimens are listed in Table 1.

Table 1 Material properties

Reinforcement			
Diameter / mm	Yield strength f_y / MPa	Ultimate strength f_u / MPa	Elongation ratio / %
Φ4	720	720	4
Φ8	300	460	38
Φ10	360	535	34
Φ12	369	520	39
Φ14[*1]	370	515	35
Φ14[*2]	310	456	31

Concrete[*3]			
Specimen	Compressive strength $f_{cu,150mm}$ / MPa	Specimen	Compressive strength $f_{cu,150mm}$ / MPa
S-RC6	28.3	P-RC6	32.5
S-RD1	28.3	P-RD1	32.5
S-ND	32.5	P-ND	32.5

Note: [*1]: Φ14 reinforcement in S-RC6/S-RD1.

[*2]: Φ14 reinforcement in other specimens.

[*3]: Concrete compressive strength is determined by testing standard cubes with a size of 150 mm × 150 mm × 150 mm, and the mean value of three cubes is taken as the compressive strength.

EXPERIMENTAL RESULTS

Seismic cyclic tests

The moment-rotation relationships of three seismic cyclic test specimens are depicted in Figures 7A to 7C. Their backbone curves are shown and compared in Figure 7D.

Experimental observations

Detailed damage progression of Specimens S-RC6 and S-RD1 can be found in Lin et al.[17] The major differences between the two specimens are: (1) flexural cracks were distributed on the frame beam of Specimen S-RC6 while its joint area was free from damage, (2) following the progressive collapse design, the flexural strength of Specimen S-RD1 was improved by approximately 30%. Consequently, when the joint rotation of Specimen S-RD1 reached ± 1.74% (i.e., beam-end displacement δ = 30 mm), diagonal cracks were found at the joint region. The joint area of Specimen S-RD1 was severely damaged and flexural cracks were found near Sections S-C and S-D on the column face after the test.

The experimental observations of Specimen S-ND in the seismic cyclic test are shown in Figure 8. When the beam end displacement (i.e., δ) reached ± 10 mm (joint rotation θ = ± 0.59%), tensile flexural cracks were found on Sections S-A and S-B on the frame beams (Figure 8A). When δ = ± 40 mm (θ = ± 2.34%), slight diagonal cracks appeared in the joint region (Figure 8B). At the displacement δ = ± 70 mm (θ = ± 4.10%), some inclined and flexural cracks matching the configuration of the new detailing were observed on the frame beams. After the specimen was cycled twice at the displacement δ = ± 100 mm, the test was terminated, and no visible damage was identified on the frame column of Specimen S-ND (Figure 8C).

The damage states of the joint regions of the three specimens are compared in Figure 9. According to the experimental results, the damage of the joint region of Specimen S-RD1 was much more severe than that of

Specimen S-RC6 because the frame beams of Specimen S-RD1 were strengthened while the adjoining columns remained unchanged after the progressive collapse design (Figure 9B). Moreover, the frame columns of Specimen S-RD1 were also damaged.[17] By adopting the proposed structural detailing, the number of diagonal cracks in the joint region of Specimen S-ND was notably reduced compared with that of Specimen S-RD1 for the same magnitude of beam-end displacement (Figure 9C), thus, indicating that the joint region damage can be considerably mitigated. Moreover, no visible damage was observed on the column face of Specimen S-ND (Figure 8C).

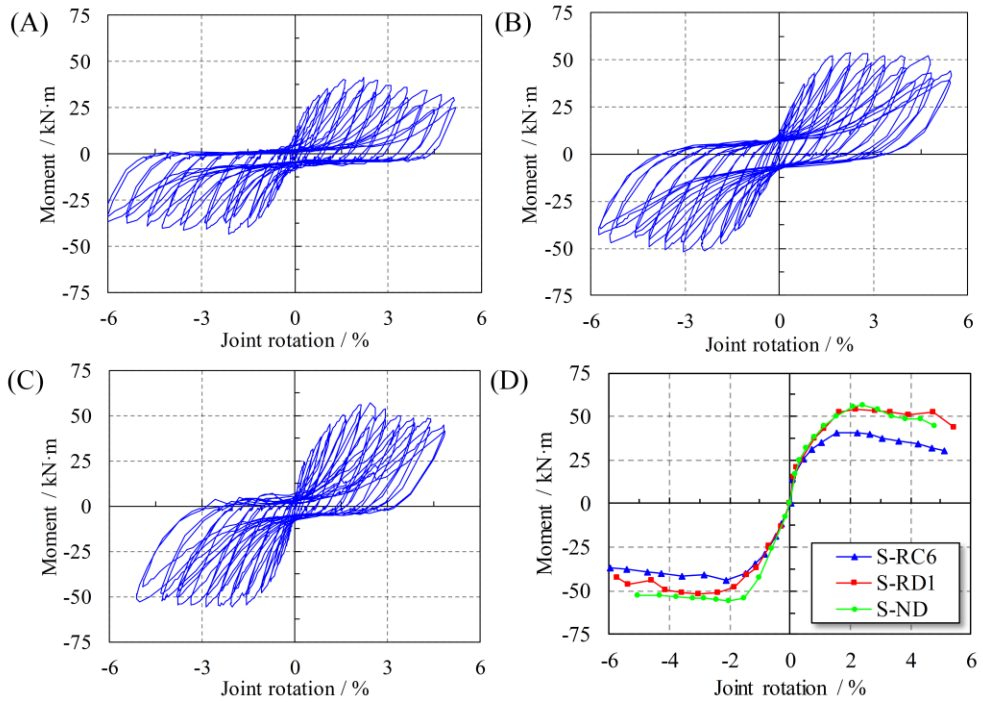

Figure 7 Moment – rotation relationships of (A) S-RC6, (B) S-RD1 and (C) S-ND; and (D) Comparison of the backbone curves.

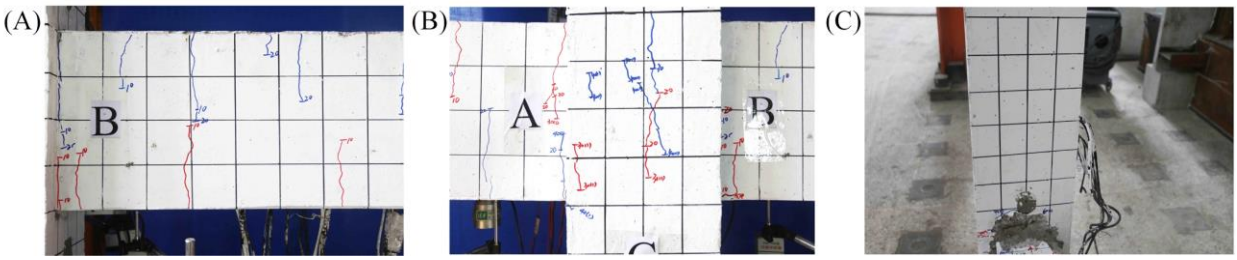

Figure 8 Damage progression of Specimen S-ND: (A) Flexural cracks on the frame beam, (B) Diagonal cracks in the joint region, and (C) Frame column was free from damage.

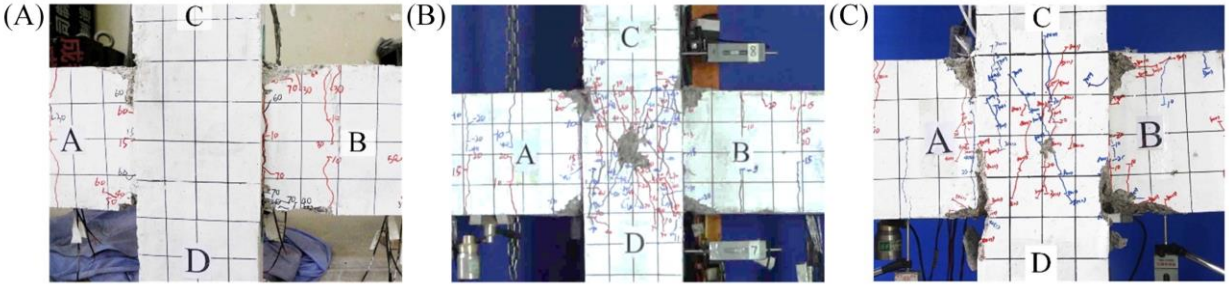

Figure 9 Comparison of joint region damages: (A) S-RC6, (B) S-RD1, and (C) S-ND.

Strain measurements

To monitor the force mechanism of the newly proposed structural detailing, strain gauges were installed at three positions of the structural detailing: a) before the rebar was bent (i.e., Gauge P1), on the bent-up/bent-down rebars (i.e., Gauge P2), and c) after the rebar was bent (i.e., Gauge P3), as shown in Figure 10. The results indicated that the newly added reinforcement with the proposed layout deformed together with the originally designed reinforcement and contributed to the sectional flexural strength at P1. At P2, the flexural contribution of the reinforcement was reduced owing to the added rebars being bent up and down. At P3, the reinforcement strain remained at a relatively low level because the added reinforcement was arranged at the mid-height of the beam at this location. It can be concluded that the flexural strength along the length of the frame beam could be adjusted by adopting the proposed detailing technique.

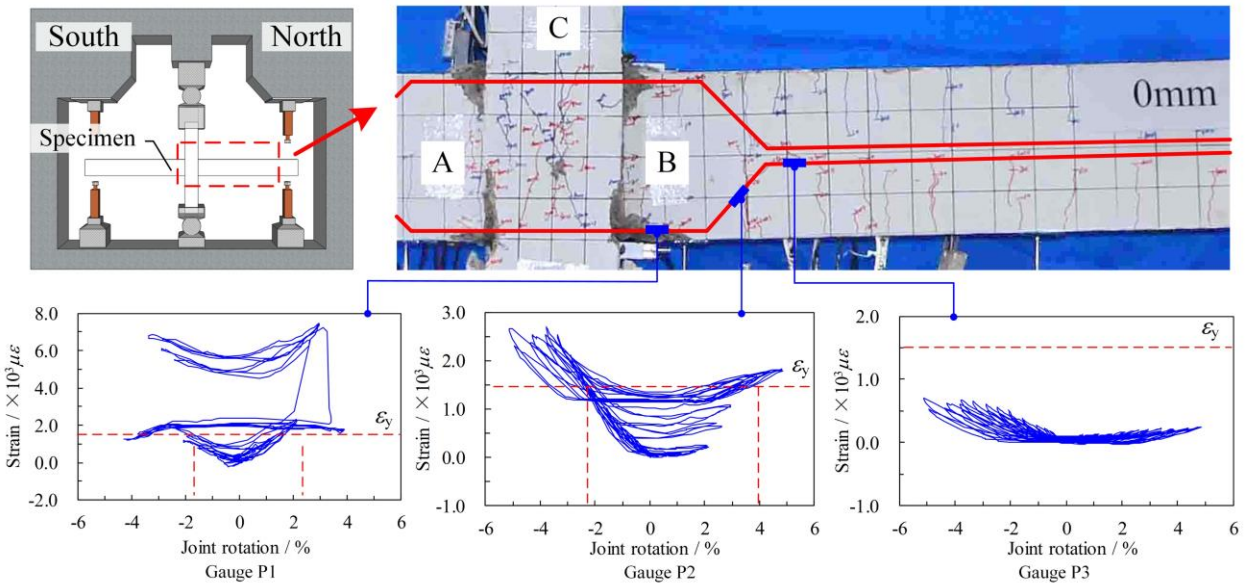

Figure 10 Strain development of the proposed structural detailing.

Progressive collapse tests

The load-displacement curves of three progressive collapse test specimens (i.e., P-RC6, P-RD1, and P-ND) are compared in Figure 11. The loading process of all the specimens can be divided into two stages, i.e., the beam and catenary mechanism stages. During the tests, the unbalanced load was resisted by the beam mechanism at the small deformation stage. At this stage, the flexural strength of the beams in conjunction with the compressive arch action developed in the specimen provided adequate resistance to progressive collapse. Moreover, the catenary action at the large deformation stage served as the ultimate resisting mechanism in progressive collapse scenarios and utilized the tensile forces from the rebars to balance the applied load. According to the DoD guideline,[10] a chord rotation of 0.20 rad (i.e., δ = 560 mm) can be used to define the deformation limit of the specimens. As such, three key points can be identified on the load-displacement curves in Figure 11: (1) the peak point of the beam mechanism (D_b, F_b), (2) the point corresponding to the chord rotation of 0.20 rad ($D_{0.20}$, $F_{0.20}$), and (3) the peak point of the catenary mechanism (D_c, F_c), where D and F denote the displacement and force, respectively. The corresponding values of the three key points are compared in Table 2 for all the three specimens. Many existing experimental studies[14-16,29] demonstrate that the displacement corresponding to rebar rupture is associated with specific randomness during the stage of catenary mechanism. Hence, the loading point ($D_{0.20}$, $F_{0.20}$) corresponding to the required chord rotation capacity (i.e., 0.20 rad) in the DoD guideline (DoD, 2016) is adopted to compare the resistance of different specimens at this stage.

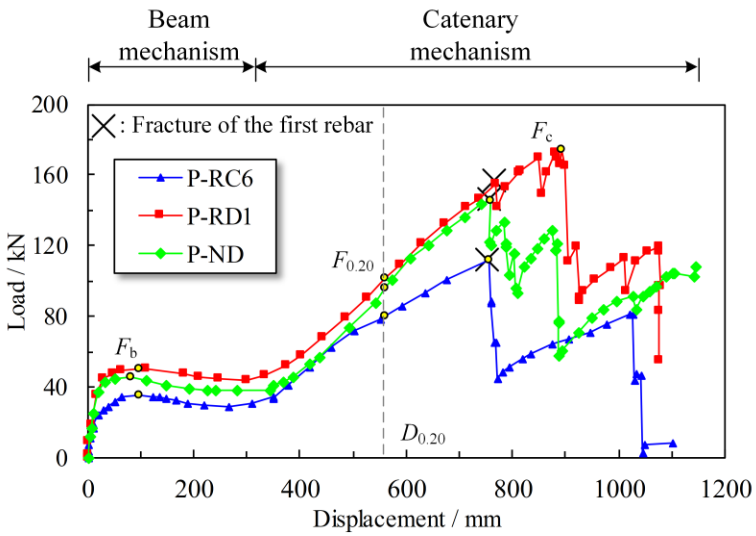

Figure 11 Comparison of load-displacement curves.

Table 2 Comparison of progressive collapse resistance of different specimens

Specimen	F_b / kN	D_b / mm	$F_{0.20}$ / kN[*1]	$D_{0.20}$ / mm	F_c / kN	D_c / mm
P-RC6	35	97	80	560	112	756
P-RD1	50	98	105	560	174	897
P-ND	45	80	99	560	146	763

Note: [*1] Required resistance at 0.20 rad is 89.5 kN.

According to the progressive collapse design of frame RC6 in Lin et al,[17] and given the consideration of the actual overstrength of the reinforcement in Table 1, the required catenary resistance is equal to 89.5 kN for all the specimens.[10] Comparison of the measured progressive collapse resistances in Table 2 indicates that Specimen P-RC6 does not meet the code requirements, whereas Specimens P-RD1 and P-ND can provide sufficient progressive collapse resistance in the catenary stages of the tests.

Experimental observations

Detailed loading processes of Specimens P-RC6 and P-RD1 can be referred to Lin et al.[17] The experimental observations of these two specimens are similar to those elicited by other comparable specimens presented in the literature.[14-16] After the progressive collapse design, the characteristic bearing capacities of P-RD1 (i.e., F_b and $F_{0.20}$) were higher than those of P-RC6. Accordingly, F_b increased by 43%, while $F_{0.20}$ increased by 31%.[17]

The loading process and final failure mode of Specimen P-ND are depicted in Figures 12 to 14. The definitions of Sections P-A to P-F are shown in Figure 4B. When the displacement of the column stub reached 50 mm, inclined flexural tensile cracks were found near the beam ends. Additionally, some minor compressive cracks were observed at the bottom of Sections P-A and P-F (Figure 12A). At the displacement of 80 mm, Specimen P-ND reached the peak resistance of the beam mechanism (i.e., F_b = 45 kN). Meanwhile, concrete crushing was found at Sections P-A and P-F. After the displacement exceeded 300 mm, the specimen entered the stage of catenary mechanism and all the sections along the beam length gradually entered into the tension state. When the displacement reached 560 mm, tensile cracks propagated along the beam height, and were found to be evenly distributed along the full length of the beam (Figure 12B). The resistance reached 99 kN at the chord rotation required in the design code (i.e., $F_{0.20}$ = 99 kN). For a displacement approximately equal to 763 mm, a rebar rupture was found at the top of Section P-F. As the displacement increased, ruptures of the longitudinal reinforcement were also found at the top of Section P-F, and the

bottom of Section P-D at the displacements of 786 mm, 806 mm, and 810 mm. Meanwhile, owing to the peeling of the concrete cover at the location of bent-up/bent-down rebars, the deformation of the newly added reinforcement could be clearly observed, as shown in Figure 12C. For a displacement of 890 mm, the reinforcement ruptured at the bottom of Section P-C and at the top of Section P-F. Afterwards, no rebar rupture was observed, and the external load continued to increase until the displacement of the column stub reached 1150 mm. After this instant, the test was terminated because the maximum displacement range of the test facility was reached. The overall deformation of Specimen P-ND during the loading process is shown in Figure 13. Moreover, the final failure mode of the specimen is presented in Figure 14.

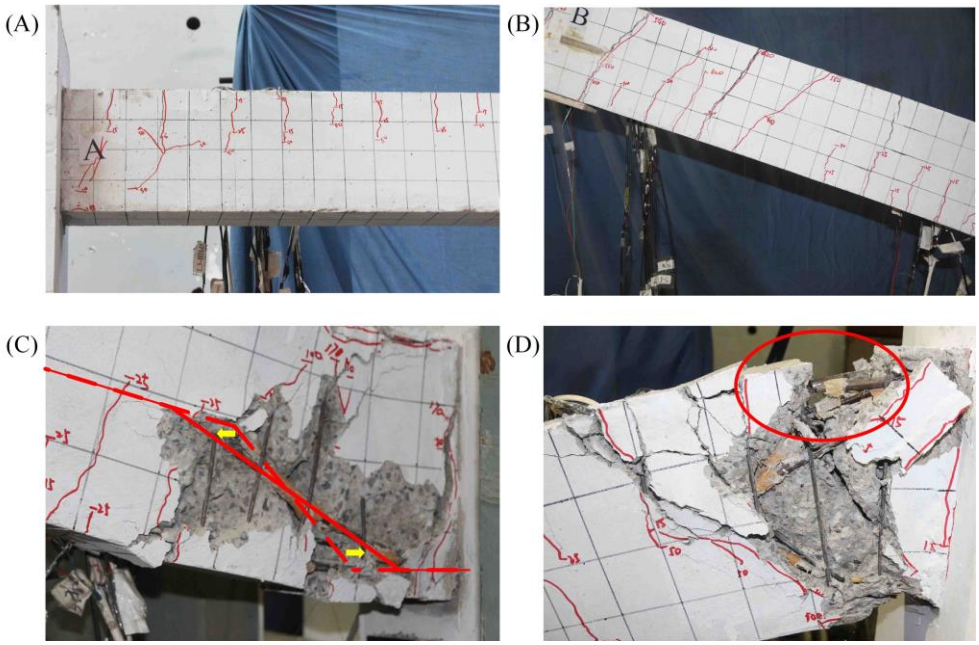

Figure 12 Progression of damage of Specimen P-ND: (A) Tensile flexural cracks at Section A, (B) Uniformly distributed tensile cracks at the catenary stage, (C) Deformation of the detailing at Section P-F, and (D) Rebar fractured at Section P-F.

By comparing the load-displacement curves of different specimens, it can be observed that the strength of Specimen P-ND is slightly smaller than that of Specimen P-RD1 after the adoption of the proposed detailing technique at the beam mechanism stage. At the catenary mechanism stage, the progressive collapse resistances are similar for the two specimens (i.e., $F_{0.20}$ = 99 kN for Specimen P-ND and $F_{0.20}$=105 kN for Specimen P-RD1). It is also worth noting that owing to the additional deformation capacity of the proposed detailing technique, the external load could still maintain a growing trend at the end of the progressive collapse test of Specimen P-ND.

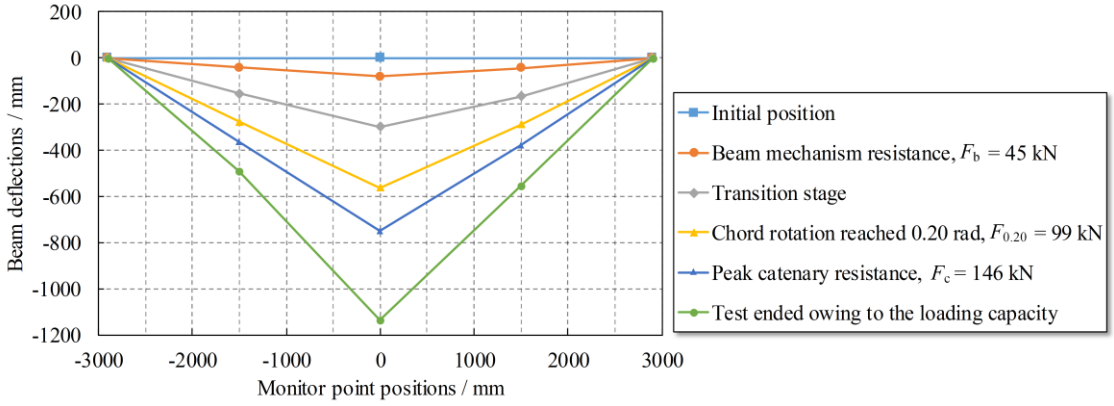

Figure 13 Overall deflections of Specimen P-ND.

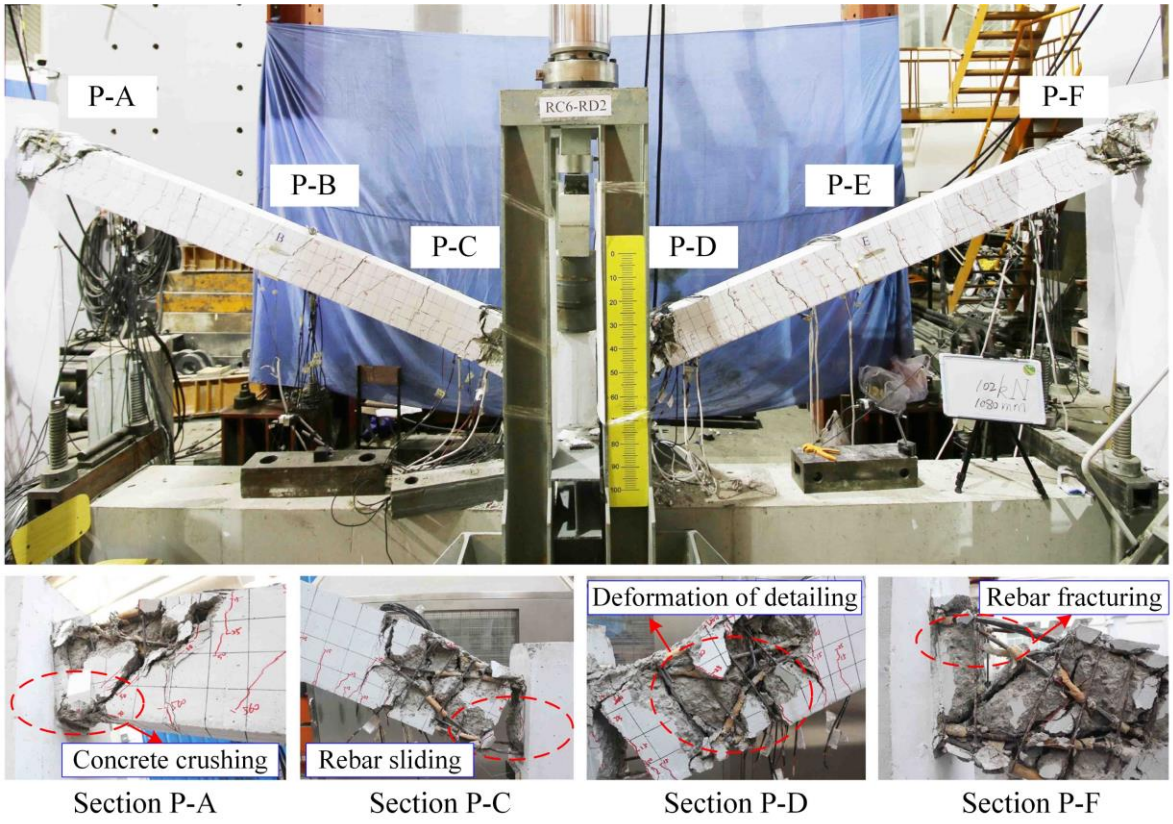

Figure 14 Failure modes of Specimen P-ND.

Strain measurements

The strain gauge measurements of Specimens P-RC6 and P-RD1 are detailed in Lin et al.[17] For Specimen P-ND, the strain development of the original seismic reinforcement at Section P-A is shown in Figure 15. The top reinforcement yielded under tension at the very initial loading stage. In contrast, the bottom reinforcement was under compression at the beam mechanism. When the loading process entered the catenary stage, the strain of the bottom reinforcement changed from compression to tension. The strain development of the original reinforcement in Specimen P-ND was identical to that of the reinforcement in Specimens P-RC6 and P-RD1 at the same location.

Figure 16 depicts the strain development of the reinforcement arranged following the proposed structural detailing technique. During the beam mechanism stage, the measured strain of the newly added anti-progressive collapse reinforcement is slightly smaller than the original reinforcement (Figure 16). This was because the reinforcement was implemented at a location closer to the mid-height of the beam. At the catenary mechanism stage, the newly added anti-progressive collapse reinforcement was under tension and contributed fully to the catenary action of the specimen. As a result, the catenary resistance of Specimen P-ND was very similar to that of Specimen P-RD1 and was significantly increased compared with Specimen P-RC6.

In summary, incorporating the proposed structural detailing, damage of the joint region and columns under the seismic action caused by the newly added anti-progressive collapse reinforcement could be notably mitigated. Moreover, in the case of the column removal scenario, the rearranged anti-progressive collapse reinforcement following the proposed detailing technique could effectively enhance the catenary action and meet the code requirements according to the DoD regulation,[10] thus, demonstrating a satisfactory performance to prevent progressive collapse of the structure.

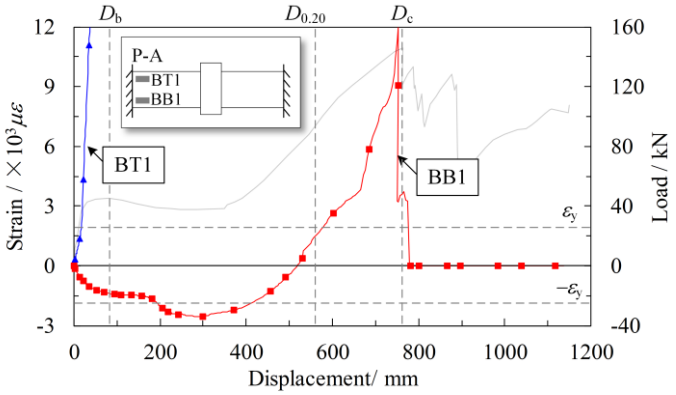

Figure 15 Strain development of original seismic reinforcement.

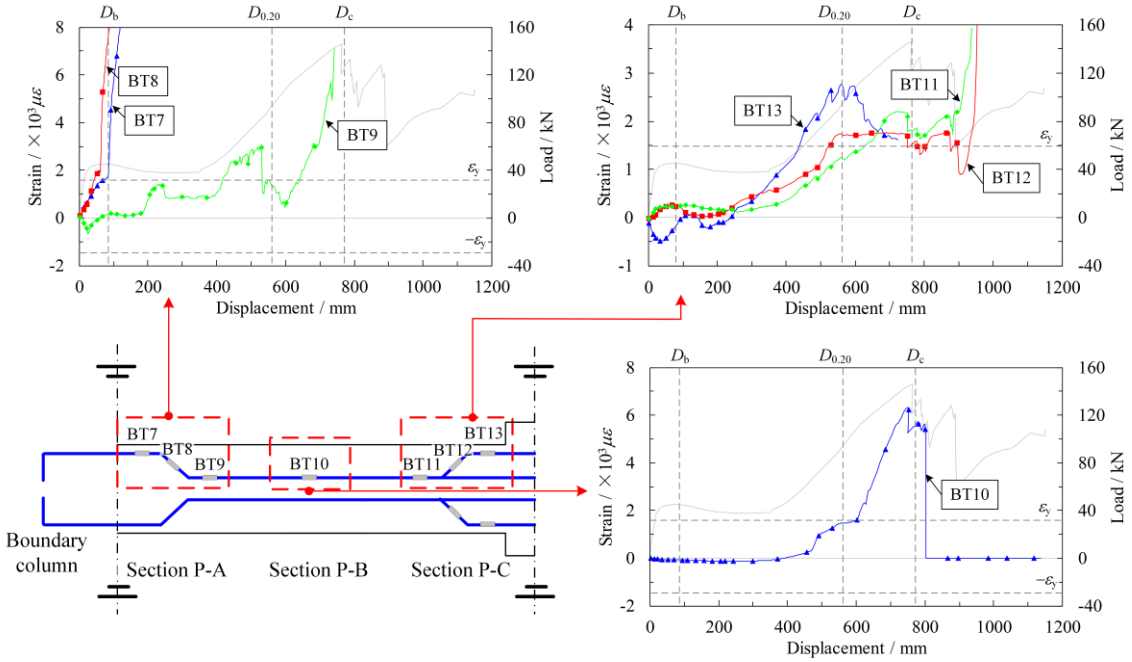

Figure 16 Strain development in added reinforcement arranged by the proposed structural detailing.

NUMERICAL VALIDATION OF THE PROPOSED STRUCTURAL DETAILING

Numerical simulations for the tests

Based on the experimental studies, fiber beam elements were used to simulate the cyclic and progressive collapse responses of the tested specimens.[33,34] The FE models used for simulating the seismic cyclic tests were established according to the method proposed by Xie et al.[30] Moreover, the progressive collapse tests were simulated by the same modeling strategy adopted in the previous studies of the authors.[19,35] The bent-up and bent-down rebars, which are arranged according to the proposed detailing technique in Specimens S-ND and P-ND, are approximately modeled by the equivalent rebar fibers located at 1/4 and 3/4 of the heights of the beam section, as shown in Figure 17.

The numerical results of all the specimens are compared with the experimental ones in Figure 18. Good agreement is shown, thus, indicating that the above modeling strategy can accurately replicate the seismic and progressive collapse behavior of both the conventional RC components and those adopting the proposed structural detailing.

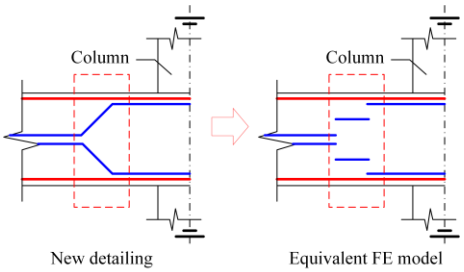

Figure 17 Modeling of the proposed structural detailing in the FE models.

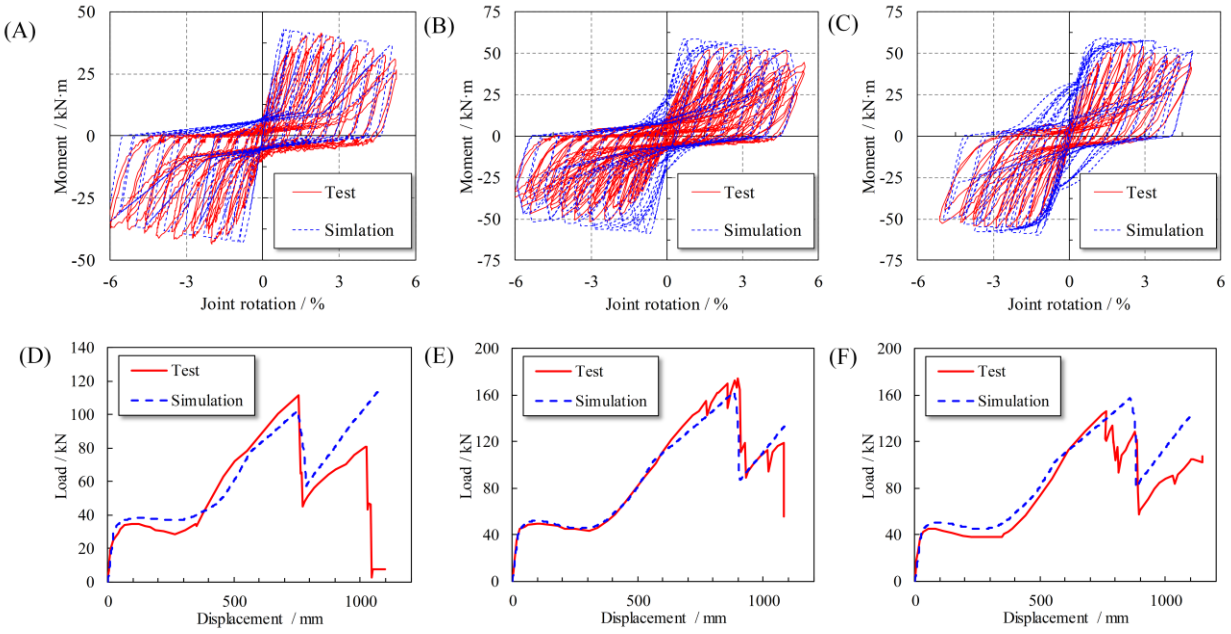

Figure 18 Comparison between numerical and experimental results for all tested specimens: (A) S-RC6, (B) S-RD1, (C) S-ND, (D)P-RC6, (E) P-RD1, and (F) P-ND.

Numerical validations of the performance of detailing at a structural level

Based on the above component-level numerical simulations of the tested specimens, the six-story RC frame in Lin et al[19] (Figure 1) is used as the prototype building to a) compare the seismic and progressive collapse performances of RC frames with different reinforcement layouts, and b) to validate the effectiveness of the proposed structural detailing. In the work of Lin et al,[19] RC6 is the original RC frame and was designed according to the Chinese design codes.[6,7] RC6-RD is the frame after implementing the progressive collapse design. According to Lin et al,[19] the RC frame might experience "strong-beam-weak-column" failure under certain ground motions because the "strong-column-weak-beam" principle was violated by RC6-RD following the progressive collapse design, which in return weakened its seismic performance. In the FE model of RC6-ND, the newly added anti-progressive collapse reinforcement is rearranged according to the layout of the proposed structural detailing. The equivalent rebar fiber modeling strategy in component-level simulations is also adopted to simulate the structural detailing in RC6-ND. Both IDA and nonlinear dynamic AP analysis are conducted to compare the seismic and progressive collapse resistances of different RC frame models (i.e., RC6, RC6-RD, RC6-ND).

Seismic performance

In the IDA, 22 far-field ground motion records in FEMA P695[36] and the widely used El-Centro 1940 record[37] were used as the ground motion input. The commonly used Rayleigh damping with a damping ratio of 5% and the widely used intensity measure PGA were used in the analysis.[7] The calculated seismic vulnerability curves of different

frames are compared in Figure 19. The results show that most collapse points of RC6-ND fall on the right-hand side of RC6 and RC6-RD. Its fragility curve is also right-shifted compared with the others, thus, indicating that the seismic performance of frame RC6-ND is improved as compared to the other two frames. For 19 ground motions among the total 23 records, the collapse PGA of RC6-ND was either larger than or equal to that of RC6 (12 records were larger and 7 records were equal to that of RC6). For the seven ground motions under which the collapse PGAs of RC-RD were smaller than those of RC6 according to Lin et al,[19] the collapse PGAs of RC6-ND were all greater than or equal to those of RC6-RD, thus, indicating a significant improvement in the seismic resistance.

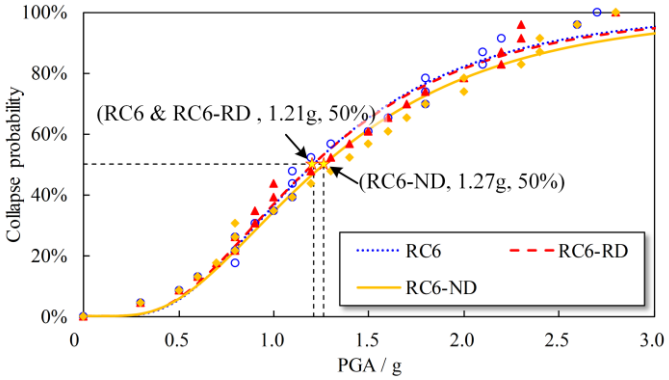

Figure 19 Collapse fragility of different RC frames.

The FRIULI-TMZ00 ground motion used in Lin et al[19] was also considered as an example. When subjected to this ground motion, the plastic hinge distributions before the onset of collapse of frames RC6-RD and RC6-ND are compared in Figure 20. After using the proposed structural detailing technique, the plastic hinge distribution of RC6-ND on the 3rd to the 5th story is more rational than that of RC6-RD. Specifically, only a limited number of plastic hinges at the beam ends are observed from Axes 2 to 4 in RC-RD. In contrast, the plastic hinges of RC6-ND are distributed more evenly at the beam and column ends, thus, indicating a better seismic energy dissipation capacity. The collapse PGA of the RC frame under the FRIULI-TMZ00 ground motion is also improved from 2.3 g (i.e., the collapse PGA of RC6-RD) to 2.6 g. Therefore, it is concluded that the proposed structural detailing could effectively mitigate the potential "strong-beam-weak-column" failure and improve the earthquake-induced collapse resistance of an RC frame after the implementation of the progressive collapse design.

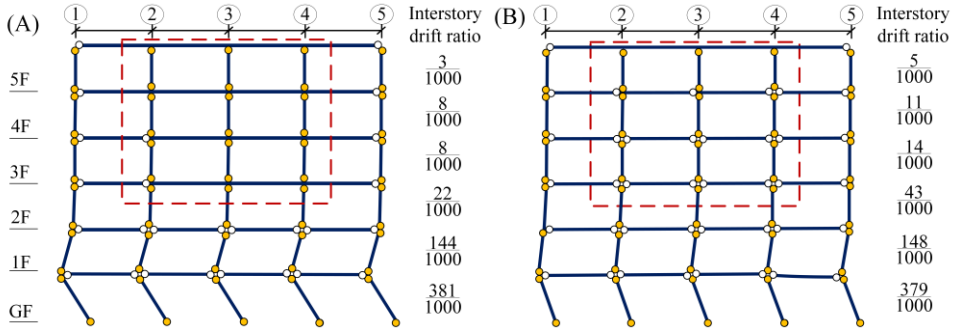

Figure 20 Plastic hinge distributions of RC6-RD and RC6-ND (Ground motion input: FRIULI-TMZ000): (A) RC6-RD (PGA = 2.3 g), and (B) RC6-ND (PGA = 2.6 g).

Progressive collapse performance

The progressive collapse resistance is evaluated following the nonlinear dynamic AP method as regulated in the DoD guidelines.[10] The dynamic responses of the RC frames under different column removal scenarios are simulated. According to Lin et al,[19] after the progressive collapse design, RC6-RD will not collapse under the design gravity load, regardless of the removal of any of the columns. The dynamic responses of RC6 and RC6-ND after removing an edge column on the first story are compared in Figure 21. It is evident that RC6 collapses while RC6-ND remains

intact. Moreover, RC6-ND is found to be as safe as RC6-RD and can be subjected to the removal of any column under the design gravity load, thus, indicating that RC6-ND also meets the progressive collapse requirements of the DoD guideline.[10]

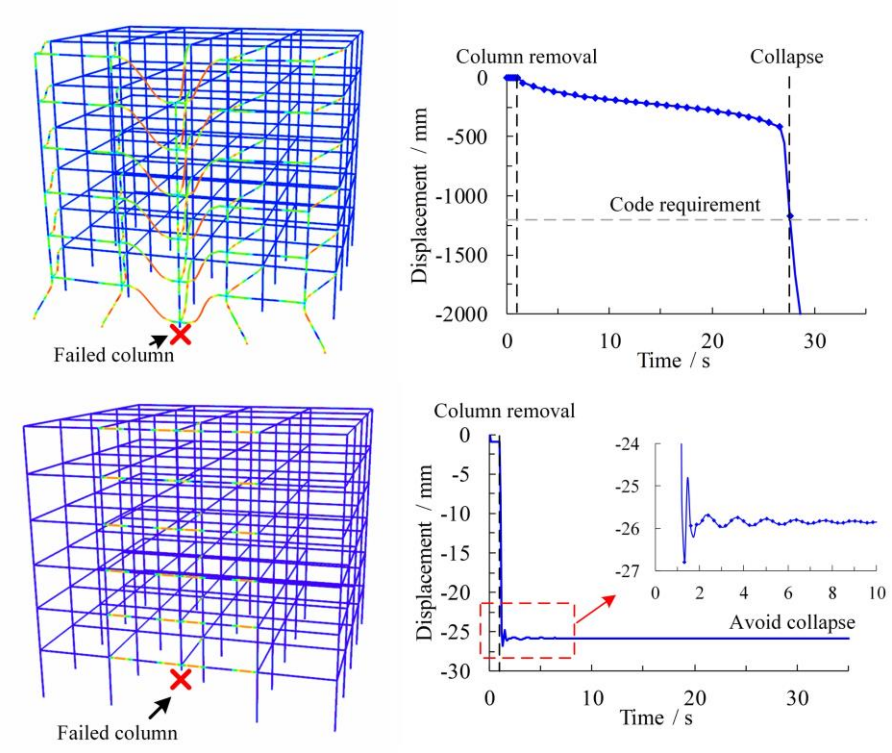

Figure 21 Responses of RC6 and RC6-ND in the case of the first story edge column removal scenario (deformation amplification factor: 1.0): (A) RC6, and (B) RC6-ND.

CONCLUSIONS

Earthquake event and progressive collapse are two critical hazards that affect the safety of RC frames. To resolve the conflict of the seismic and progressive collapse designs of RC frames, a novel structural detailing is proposed. Both seismic cyclic and progressive collapse tests of RC substructures were conducted to verify the effectiveness of the proposed structural detailing. Based on the experimental results, numerical simulations were undertaken using validated FE models to examine the seismic and progressive collapse resistances of the RC frame adopting the proposed structural detailing. The main conclusions of this study are:

(1) The experimental results showed that although the existing progressive collapse design can effectively enhance the progressive collapse resistance of RC frames, the newly added reinforcement in the beams can make the joint region and columns more vulnerable under the seismic cyclic load. After adopting the proposed structural detailing, earthquake-induced damage at the joint region and columns can be significantly mitigated.

(2) In the progressive collapse tests of the RC substructures under a middle-column removal scenario, the newly added anti-progressive collapse reinforcement that was rearranged according to the proposed detailing could effectively improve the catenary resistance of the specimen, with a similar level of improvement to that of the specimen incorporating the conventional progressive collapse design, thereby satisfying the code requirements.

(3) The FE modeling strategy of the proposed detailing was developed and validated against experimental results. According to the IDA and nonlinear dynamic AP analysis of a six-story RC frame, and compared with the RC frame redesigned according to the existing progressive collapse design codes, the RC frame with the new structural detailing was found to provide better seismic performance without increasing overall material consumption.

Meanwhile, the progressive collapse resistance of the frame also satisfies the requirement of the design guidelines.[5,10,11]

The outcome of this study is expected to provide a reference for the future development of the multi-hazard design philosophy for RC frames against various concurrent hazards.

ACKNOWLEDGEMENT

The authors are grateful for the financial support received from the National Natural Science Foundation of China (No. 51778341). The authors would also like to acknowledge Prof. Youlin Xu from The Hong Kong Polytechnic University for his helpful suggestions for this work.

REFERENCE

1. Sozen MA, Thornton CH, Corley WG, Sr PFM. The Oklahoma City bombing: structure and mechanisms of the Murrah Building. *Journal of Performance of Constructed Facilities.* 1998; 12(3): 120-136. https://doi.org/10.1061/(ASCE)0887-3828(1998)12:3(120)
2. Lu XZ, Ye LP, Ma YH, and Tang DY. Lessons from the collapse of typical RC frames in Xuankou school during the great Wenchuan Earthquake. *Advances in Structural Engineering.* 2012; 15(1): 139-153. https://doi.org/10.1260/1369-4332.15.1.139
3. ACI 318-14. Building code requirements for structural concrete (ACI 318-14) and commentary (318R-14). Detroit, MI: American Concrete Institute, 2014.
4. CEN. Eurocode8: Design provisions for earthquake resistance of structure (EN 1998-1:2004). Brussels: European Committee for Standardization, 2004.
5. CECS. Code for anti-collapse design of building structures (CECS 392: 2014). Beijing: China Association for Engineering Construction Standardization, 2014. (in Chinese)
6. MOHURD. Code for design of concrete structures (GB50010-2010). Beijing: Ministry of Housing and Urban-Rural Development of the People's Republic of China, 2010. (in Chinese)
7. MOHURD. Code for seismic design of buildings (GB50011-2010). Beijing: Ministry of Housing and Urban-Rural Development of the People's Republic of China, 2010. (in Chinese)
8. Ellingwood BR. Mitigating risk from abnormal loads and progressive collapse. *Journal of Performance of Constructed Facilities.* 2006; 20(4): 315-323. https://doi.org/10.1061/(ASCE)0887-3828(2006)20:4(315)
9. Marjanishvili SM. Progressive analysis procedure for progressive collapse. *Journal of Performance of Constructed Facilities.* 2004; 18(2): 79-85. https://doi.org/10.1061/(ASCE)0887-3828(2004)18:2(79)
10. DoD. Design of structures to resist progressive collapse (Unified facility criteria, UFC 4-023-03), Washington, DC: Department of Defense, 2016.
11. GSA. Alternate path analysis and design guidelines for progressive collapse resistance (GSA 2016). Washington, DC: General Services Administration, 2016.
12. Yi WJ, He QF, Xiao Y, Kunnath SK. Experimental study on progressive collapse-resistant behavior of reinforced concrete frame structures. *ACI Structural Journal.* 2008; 105(4): 433. doi: 10.14359/19857
13. Su YP, Tian Y, Song XS. Progressive Collapse Resistance of Axially-Restrained Frame Beams. *ACI Structural Journal.* 2009, 106(5). doi: 10.14359/51663100
14. Yu J, Tan KH. Structural behavior of RC beam-column subassemblages under a middle column removal scenario. *Journal of Structural Engineering.* 2012; 139(2): 233-250. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000658
15. Qian K, Li B, Ma JX. Load-carrying mechanism to resist progressive collapse of RC buildings. *Journal of Structural Engineering.* 2015; 141(2): 04014107. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001046
16. Ren PQ, Li Y, Lu XZ, Guan H, Zhou YL. Experimental investigation of progressive collapse resistance of one-way reinforced concrete beam-slab substructures under a middle-column-removal scenario. *Engineering Structures.* 2016; 118: 28-40. https://doi.org/10.1016/j.engstruct.2016.03.051
17. Lin KQ, Lu XZ, Li Y, Guan H. Experimental study of a novel multi-hazard resistant prefabricated concrete frame structure. *Soil Dynamics and Earthquake Engineering.* 2018; In press. https://doi.org/10.1016/j.soildyn.2018.04.011
18. Li Y, Ahuja A, Padgett JE. Review of methods to assess, design for, and mitigate multiple hazards. *Journal of Performance of Constructed Facilities.* 2012; 26(1): 104-117. https://doi.org/10.1061/(ASCE)CF.1943-5509.0000279

19. Lin KQ, Li Y, Lu XZ, Guan H. Effects of seismic and progressive collapse designs on the vulnerability of RC frame structures. *Journal of Performance of Constructed Facilities.* 2017; 31(1): 04016079. https://doi.org/10.1061/(ASCE)CF.1943-5509.0000942
20. Feng P, Qiang HL, Qin WH, Gao M. A novel kinked rebar configuration for simultaneously improving the seismic performance and progressive collapse resistance of RC frame structures. *Engineering Structures*. 2017; 147: 752-767. https://doi.org/10.1016/j.engstruct.2017.06.042
21. Eom TS, Park HG, Hwang HJ, Kang SM. Plastic hinge relocation methods for emulative PC beam-column connections. *Journal of Structural Engineering.* 2016; 142(2): 04015111. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001378
22. Qu Z, Kishiki S, Maida Y, Sakata H. Subassemblage cyclic loading tests of buckling-restrained braced RC frames with unconstrained gusset connections. *Journal of Structural Engineering.* 2015; 142(2): 04015128. https://doi.org/10.1061/(ASCE)ST.1943-541X.0001407
23. Sasmal S, Nath D. Evaluation of performance of non‐invasive upgrade strategy for beam–column sub‐assemblages of poorly designed structures under seismic type loading. *Earthquake Engineering & Structural Dynamics.* 2016; 45(11): 1817-1835. https://doi.org/10.1002/eqe.2730
24. Yu J, Tan KH. Special detailing techniques to improve structural resistance against progressive collapse. *Journal of Structural Engineering.* 2013; 140(3): 04013077. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000886
25. Alogla K, Weekes L, Augusthus-Nelson L. A new mitigation scheme to resist progressive collapse of RC structures. *Construction and Building Materials.* 2016; 125: 533-545. https://doi.org/10.1016/j.conbuildmat.2016.08.084
26. Li M, Sasani M. Integrity and progressive collapse resistance of RC structures with ordinary and special moment frames. *Engineering Structures.* 2015; 95: 71-79. https://doi.org/10.1016/j.engstruct.2015.03.050
27. Livingston E, Sasani M, Bazan M, Sagiroglu S. Progressive collapse resistance of RC beams. *Engineering Structures.* 2015; 95: 61-70. https://doi.org/10.1016/j.engstruct.2015.03.044
28. Li Y, Lu XZ, Guan H, Ye LP. Progressive collapse resistance demand of RC frames under catenary mechanism. *ACI Structural Journal.* 2014; 111(5):1225-1234. doi: 10.14359/51687029.
29. Lu XZ, Lin KQ, Li Y, Guan H, Ren PQ, Zhou YL. Experimental investigation of RC beam-slab substructures against progressive collapse subject to an edge-column-removal scenario. *Engineering Structures.* 2017; 149: 91-103. https://doi.org/10.1016/j.engstruct.2016.07.039
30. Xie LL, Lu XZ, Guan H, Lu X. Experimental study and numerical model calibration for earthquake-induced collapse of RC frames with emphasis on key columns, joints, and the overall structure. *Journal of Earthquake Engineering.* 2015; 19(8): 1320-1344. https://doi.org/10.1080/13632469.2015.1040897
31. Tsonos AG. Cyclic load behavior of reinforced concrete beam-column sub-assemblages of modern structures. *ACI Structural Journal.* 2007; 104(4): 468-478. doi: 10.14359/18777
32. Mirzabagheri S, Tasnimi AA, Mohammadi MS. Behavior of interior RC wide and conventional beam-column roof joints under cyclic load. *Engineering Structures.* 2016; 111: 333-344. https://doi.org/10.1016/j.engstruct.2015.12.011
33. Spacone E, Filippou FC, Taucer FF. Fibre beam-column model for non‐linear analysis of R/C frames: Part I. Formulation. *Earthquake Engineering & Structural Dynamics*. 1996; 25(7): 711-725. https://doi.org/10.1002/(SICI)1096-9845(199607)25:7<711::AID-EQE576>3.0.CO;2-9
34. Spacone E, Filippou F C, Taucer F F. Fibre beam-column model for non‐linear analysis of r/c frames: Part II. Applications. *Earthquake engineering & structural dynamics*. 1996; 25(7): 727-742. https://doi.org/10.1002/(SICI)1096-9845(199607)25:7<727::AID-EQE577>3.0.CO;2-O
35. Lu XZ, Lin KQ, Li CF, Li Y. New analytical calculation models for compressive arch action in reinforced concrete structures. *Engineering Structures.* 2018; 168: 721-735. https://doi.org/10.1016/j.engstruct.2018.04.097
36. FEMA. (2009). Quantification of building seismic performance factors (FEMA P695). Washington, DC: Federal Emergency Management Agency, 2009
37. Chopra AK. Dynamics of structures: theory and applications to earthquake engineering. Second Edition. Prentice Hall, Upper Saddle River, New Jersey, 245-249. 2001.