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Numerical Evaluation of the Reliability of Using Single-Story Substructures to Study Progressive Collapse Behaviour of Multi-Story RC Frames

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Abstract: Progressive collapse is a global failure for a multi-story building. All stories above the removed column will consequently deform and help redistribute the loads initially withstood by the removed column. However, due to cost and excessive time to be involved, the majority of existing experimental researches regarding progressive collapse rely on single-story beam-column substructures or sub-assemblages. To date, how to use the results from single-story substructures or sub-assemblages to fully or confidently study the behavior of multi-story building is still unclear. Thus, it is imperative to investigate the relationship between the results of single-story substructures and the real behavior of multi-story buildings. Thus, for this purpose, in the present study, a series of planar multi-story reinforced concrete (RC) beam-column substructures were modeled using high-fidelity finite element software LS-DYNA. Firstly, the numerical models were validated by the test results of two three-story planar substructures with different design spans. Secondly, the validated models were explored on various load resistance of each story in the investigated multi-story frame. In addition, the effects of boundary conditions, missing column locations, story numbers on the variation of load resistance were studied in detail using the models.

Keywords: Progressive collapse; Multi-Story RC frames; Load transfer mechanism; Numerical simulations; Column removal scenario

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1 **1. Introduction**

2

3 Progressive collapse is defined in ASCE/SEI 7 [1] as “the spread of an initial local failure from
4 element to element, eventually resulting in the collapse of an entire structure or a disproportionately
5 large part of it”. To date, there are two main methods to design buildings to mitigate progressive
6 collapse: direct and indirect design methods. For **the** indirect design method, **integrity**, redundancy,
7 ductility, and minimum tie-force are required. However, when local **damages** are triggered, it is
8 difficult to quantitatively evaluate the capacity and behavior of remaining building in resisting
9 progressive collapse based on this method. For the direct design method, alternative load path method
10 is most commonly used as it is event-independent. To understand the behavior of multi-story buildings
11 subjected to sudden column missing scenario, Sasani et al. [2] carried out an on-site test for an actual
12 10-story reinforced concrete building following the explosive damage of an exterior column. Similarly,
13 a six-story RC infilled-frame building was evaluated following the removal of two adjacent exterior
14 columns simultaneously by Sasani [3]. Song et al. [4] tested a steel frame building subjected to
15 physically removal of four ground columns from one of the perimeter frames to study the load
16 redistribution of the building after each column removal. However, as the service load (live load and
17 **partial dead load**) was removed prior to on-situ tests, the measured displacement response was little.
18 Majority of on-situ tests only experienced elastic response. The plastic behavior especially for
19 compressive arch action (CAA) and tensile catenary action (TCA) could not be captured and therefore
20 be evaluated in detail. Thus, **the** majority of existing tests in laboratory regarding progressive collapse
21 were single-story beam-column substructures based on alternative load path method (relied on **push-**
22 **down** loading regime). A number of tests [5-13] quantified the effects of geometric characteristic and
23 reinforcing details on the mobilization of CAA and TCA for progressive collapse prevention. It was
24 found that the span/depth ratio has great effects on the mobilization of CAA when the RC frame

1 subjected to the loss of a middle column scenario. In addition, the CAA capacity is sensitive to the
2 horizontal stiffness provided by the beam ends. However, the amount of longitudinal reinforcements in
3 the structural concrete members has little effect on developing CAA. The researchers [8] indicated that
4 the continuous top longitudinal reinforcements contributed to TCA capacity while Yi et al. [14]
5 indicated that both top and bottom longitudinal reinforcements provided contributed to TCA capacity.

6 Moreover, several studies [15-17] were carried out to evaluate the dynamic response and dynamic
7 load increase factor of RC beam-column sub-assemblages subjected to sudden column removal
8 scenario. Qian and Li [15] indicated that the acceleration of the frame after sudden column removal
9 could be as large as 3.5g, where g is the acceleration of gravity, and the dynamic load increase factor
10 could be less than 1.38. Qian and Li [16] quantified the slab effects on the dynamic response of RC
11 frames subjected to the sudden removal of a ground corner column. In addition, they proposed an
12 equivalent single-degree-of-freedom (SDOF) model to predict the dynamic ultimate load capacity of
13 the tested specimens. The dynamic load increase factor of tested specimens was ranged from 1.30 to
14 1.34. Liu et al. [17] investigated the dynamic behavior of steel frames with different connections
15 subjected to sudden removal of a center column experimentally. The test results indicated that the
16 dynamic phenomenon may detriment the behavior of steel connections and degrade the progressive
17 collapse resistance of the substructures.

18 However, the reliability of using single-story substructures to study the behavior of multi-story
19 buildings is based on the assumption that the stories above the removed column have identical
20 performance, which is questionable and has not been proved. Weng et al. [18] used high fidelity finite
21 element (FE) models to investigate the load resisting mechanisms of each story for a multi-story flat
22 slab structure under a middle column loss scenario. The numerical results indicated that the load
23 resistance from each story in a multi-story flat slab building was different and the largest load resisting

1 capacity occurred in the first story. However, for a multi-story RC frame, it may be different to flat
2 slab structures as beams could help to redistribute the loads. In Qian and Li [19], two multi-story
3 frames were tested based on displacement-controlled push-down method. As it is not feasible to test a
4 multi-story frame to assess the load resisting contribution of each story, FE models, which are
5 validated against the experimental tests, are used for deeper understanding of the various load transfer
6 mechanism and load resisting contribution of each floor consequently. Moreover, the effects of the
7 number of stories, boundary conditions, and missing column locations are also studied using the
8 validated numerical models.

9 **2. Previous Experimental Work**

10 A quasi-static experimental study on progressive collapse resistance of planar RC beam-column
11 substructures subjected to push-down loading regimes was conducted by Qian and Li [19] and the test
12 results of two bare frames are used to validate the reliability of FE models in this numerical study.
13 These two specimens (BFS and BFL) were one-quarter scaled. They were assumed to be subjected to
14 the loss of a penultimate column. The dimension and reinforcement details of Specimen BFS are
15 shown in Fig. 1. As can be seen in the figure, the beam span was 1800 mm. The story height was 900
16 mm in the first story and 825 mm in upper stories. The cross section of the beam and column was 90
17 mm×140 mm and 150 mm×150 mm, respectively. The concrete clear cover was 7 mm and 10 mm for
18 beam and column, respectively. Enlarged foundation base with a size of 400 mm×300 mm was
19 designed at the toe of side columns for fixing. The hoop stirrups with 90 degrees bends were utilized
20 for transverse reinforcements. As it was non-seismically designed, no transverse reinforcements were
21 placed in the joint region. The curtailment of longitudinal reinforcements in the beam was in
22 accordance with Singapore Code CP-65 [20]. The middle column in ground level is assumed removed
23 before test and thus, the middle column was only fabricated the upper two stories. For Specimen BFL,

1 similar reinforcement details and dimensions to Specimen BFS were used except longer span of 2400
2 mm designed. Average cylinder compressive strength measured on the days of testing for both
3 specimens was 32.1 MPa. The yield strength of R3, R6, and T10 were 417 MPa, 449 MPa, and 515
4 MPa, respectively. The ultimate strength of R3, R6, and T10 were 479 MPa, 537 MPa, and 594 MPa.
5 The measured elongation ratio of R3, R6, and T10 were 9.7 %, 13.3 %, and 16.9 %, respectively. “R”
6 represents plain reinforcement while “T” represents deformed reinforcement.

7 The typical experimental setup and locations of instrumentations are shown in Fig. 2. As
8 shown in the figure, the specimens were fixed to the strong floor by the foundation bases, which were
9 cast monolithically with the side columns. A steel column and a specially designed steel assembly
10 were installed to avoid unforeseen out-of-plane movement of the specimen. This specially designed
11 steel assembly only allows vertical movement of the middle column through constraining its rotational
12 and horizontal movements. A hydraulic jack with a 600 mm stroke was installed on the steel column to
13 apply vertical load. It should be noted that the displacement-controlled push-down loading method was
14 adopted in the reference tests [19]. A load cell was installed above the hydraulic jack to measure the
15 applied vertical load. A roller together with a tension/compression load cell was installed horizontally
16 at each extension part of the specimen to simulate the horizontal constraints of the beams in the
17 surrounding bay.

18 **3. Numerical Model Development and Validation**

19 To illustrate the variation of load resisting capacity and mechanisms of the beams in different
20 stores overtly, explicit solver software LS-DYNA [21] was used to build the FE models due to its
21 numerical stability and various constitutive models available. The FE models were validated based on
22 the experimental results first. As shown in Fig. 3, similar boundary conditions as experimental tests
23 were used at the FE models. As gaps were deliberately left near the horizontal constraints for the

1 **facility** of installation, springs were installed horizontally at the beam ends of the extension part. The
2 stiffness of the spring was determined by the measured horizontal reaction force and horizontal
3 movements at the ends, which was roughly equal to 80 kN/mm at each beam end with a total of 240
4 kN/mm.

5 ***3.1 Element types***

6 Fig. 3 shows the numerical model of Specimen BFS. Concrete is simulated by 8-node solid
7 elements with reduced integration scheme. This solid formulation only has one integration point in
8 each element, which can enhance computational efficiency with **the promise** of sufficient accuracy, but
9 hourglass control should be defined properly when this type of **element** was adopted. Moreover,
10 reinforcing bars were modeled using 2-node Hughes-Liu beam elements with 2×2 Gauss quadrature
11 integration at the cross-section. This beam formulation can simulate the behavior of axial force, bi-
12 axial bending, and finite transverse shear strains [22]. Furthermore, the rigid plates for supports or
13 loading points were also modeled by 8-node solid elements and the springs for horizontal restraints
14 were simulated by discrete elements.

15 ***3.2 Bond-slip relationship simulation***

16 To improve the accuracy of **modeling**, considering **the** bond-slip relationship between
17 reinforcement and concrete is important, as the perfect bonding assumption used in other models will
18 cause over-prediction of **load-carrying** capacity and lead to premature fracture of reinforcement due to
19 stress concentration [23]. In this numerical study, the bond-slip relationship between reinforcement
20 and concrete material was considered by using keyword *CONTACT_1D to define one-dimensional

1 contact interface between the concrete and rebar elements. Virtual springs are defined between the
2 slave nodes from beam elements and the master nodes from solid elements, and the spring force
3 depends on relative displacements between the slave and master nodes [21]. However, simulating
4 bond-slip behavior for all reinforcements would be complicated and required more computational
5 resources. Based on test results, it was noted that the slip occurred mainly at the beam-column joints
6 and the curtail point of top beam longitudinal reinforcements. As a result, similar to previous studies
7 [24, 25], the CONTACT_1D function was only used for the beam longitudinal reinforcements at the
8 location of beam-column joints and the reinforcements near curtail points, as shown in Fig. 3. Besides,
9 the remaining reinforcements were assumed to be perfect bonding to concrete using keyword
10 *Constrained_Lagrange_In_Solid. To calibrate the properties of Contact_1D, the bond-slip
11 relationship proposed by fib Model Code 2010 [26] was applied. For monotonic loading, the bond
12 stress τ_b between concrete and hot-rolled plain bar for pull-out failure can be calculated as following
13 [26]:

$$14 \quad \begin{cases} \tau_b = \tau_{b\max} (s / s_1)^{0.5} & \text{for } 0 \leq s \leq s_1 \\ \tau_b = \tau_{b\max} = 0.3\sqrt{f_c} & \text{for } s > s_1 \end{cases} \quad (1)$$

15 Where $s_1 = 0.1$ mm; f_c is standard cylinder compressive strength. In the tests, the compressive
16 strength of concrete is 32.1 Mpa. Therefore, $\tau_{b\max} = 1.70$ MPa .

17 For the CONTACT_1D (LS-DYNA) function [21], the bonding relationship between the beam
18 elements and solid elements is assumed to be elastic-perfectly-plastic. After elastic stage, the bond
19 stress would decay following an exponential damage curve. The constitutive law of shear stress τ and
20 slip s is given as [21]:

$$21 \quad \begin{cases} \tau = G_s s & \text{for } s \leq s_{\max} \\ \tau = \tau_{\max} e^{-h_{\text{dmg}} D} & \text{for } s > s_{\max} \end{cases} \quad (2)$$

1 where G_s is bond shear modulus; s_{\max} is maximum elastic slip; h_{dmg} is damage curve exponential
2 coefficient; D is damage parameter, which is equals $(s-s_{\max})$.

3 The comparison between Eqs. (1) and (2) indicates that the values of h_{dmg} and τ_{\max} are equal to 0
4 and 1.7 Mpa, respectively. Based on the suggestion from Pham et al. [24] and Yu et al. [25], $s_{\max} = 0.5$
5 $s_I = 0.05$ mm. Therefore, $G_s = \tau_{\max} / s_{\max} = 34 \text{MPa} / \text{mm}$. The comparison of the bond-slip relationship
6 between the fib Model Code 2010 [26] and the suitable model used in CONTACT_1D is shown in Fig.
7 4.

8 **3.3 Material model**

9 In this study, Continuous surface cap model (CSCM) is chosen to simulate concrete material. This
10 model can effectively model damage-based softening and modulus reduction, shear dilation, shear
11 compaction, confinement effect, and strain rate effect under low constraint conditions [27]. Previous
12 studies had proven its accuracy in the simulation of RC components under both quasi-static and
13 dynamic conditions [24, 25, 28].

14 The CSCM provides a simplified version (*Mat_CSCM_CONCRETE) for concrete materials
15 with the compressive strength between 28 Mpa and 48 Mpa. The default parameters depend on three
16 input parameters: unconfined compressive strength f'_c , maximum aggregate size A_g , and units. For both
17 Specimens BFS and BFL, f'_c and A_g are 32.1 Mpa and 8 mm, respectively. The CSCM also provides a
18 strain-based approach of erosion algorithm to simulate material failure, and the related parameter is
19 “ERODE”. When the “ERODE” is set greater than 1.0, the concrete elements would be deleted if
20 damage index exceeds 0.99 and the maximum principal strain exceeds (ERODE-1.0) according to LS-
21 DYNA keyword user’s manual [21]. This feature is used here to effectively model the failure mode of
22 the frame. In the reference test results, the failure modes of the multi-story frames were governed by

1 the flexure and tensile actions, primarily denoted by the formation of severe cracks at the beam ends
2 near the center column and at the curtail points of beam top longitudinal reinforcements. Therefore, the
3 maximum principal strain is a suitable criterion for erosion algorithm. The value of “ERODE” is mesh-
4 dependent, and set as 1.10, corresponding to the maximum principal of 0.1, for element size 20 mm
5 according to the previous work [24]. The strain rate effect of the CSCM is ignored because only quasi-
6 static behavior is considered.

7 The isotropic elastic-plastic material model Mat_Plastic_Kinematic (MAT_003) is used to model
8 reinforcements. The elastic modulus, yield strength, tangential modulus after yielding, and ultimate
9 strain is determined based on properties of steel bars. Also, the strain rate effect is excluded.

10 As choosing the appropriate mesh size is important to obtain reliable and effective results, mesh
11 sensitivity is evaluated. Four different mesh sizes of elements (side length for solid elements and
12 length for beam elements), including 30 mm, 25 mm, 20 mm, and 15 mm, were employed for
13 Specimen BFS. The results of the load-removed column displacement (RCD) relationship for different
14 mesh sizes are shown in Fig. 5. Obviously, mesh size of 20 mm is adequate, as further mesh
15 refinement is not able to cause any remarkable convergence but instead taking larger computational
16 resources. As a result, the mesh size is chosen as 20 mm.

17 However, based on the default parameters of the CSCM, the numerical models will overestimate
18 the initial stiffness and load resisting capacity of the specimens, as shown in Fig. 6. Unconfined
19 uniaxial stress-strain relationship for 32.1 MPa based on the default parameters of the CSCM is shown
20 in Fig. 7. As can be seen from the figure, the compressive strength is attained at a strain of 0.001. But
21 in reality, when normal strength concrete reaches its compressive strength, the strain is usually at about
22 0.002. Therefore, the elastic modulus of concrete should be reduced properly to improve the numerical
23 results, which had been pointed out by Yu et al. [25]. On the other hand, Yu et al. [25, 28] suggested

1 that the tensile fracture energy G_{ft} could be reduced to 80 % of the default one when the simulating
2 **result** is over predicted. If shear or compressive based damage is significant, then setting G_{fs}
3 (compressive fracture energy) = $0.5 G_{ft}$ and G_{fc} (shear fracture energy) = $50 G_{ft}$ is reasonable [25].
4 However, the default ones are assumed as $G_{fs}=G_{ft}$ and $G_{fc}=100G_{ft}$. Since severe shear cracks were
5 formed in the exterior joint in the first floor and the concrete crushing was not obvious during testing,
6 only the reduced shear fracture energy is used for **the** CSCM model herein. The user-specified material
7 property inputs for CSCM are listed in Table 1. When the adjusted material property is used for
8 simulating, the stiffness of the unconfined uniaxial compression is lower than that of the default one
9 and the compressive stress reduces faster in the softening stage, as shown in Fig. 7. However, the
10 adjusted material property can improve the numerical results significantly, as shown in Fig. 6.
11 Therefore, these adjustments are finally used to simulate Specimens BFS and BFL.

12 **3.4 Verification of numerical model**

13 Fig. 8 shows the comparison of load-displacement curves from numerical simulation and
14 experimental results. Generally, the FE models can simulate all three stages of the structural responses
15 well. In the first stage, the structural resistance increases until reaching the first peak load. The
16 resistance is attributed into the flexural action and compressive arch action (CAA). Then, the
17 resistance decreases due to the weakening of CAA in the second stage. In the last stage, the resistance
18 increases again due to the development of tensile catenary action (TCA), and abrupt reduction since
19 rebar fracture was also simulated. For Specimens BFS and BFL, **the error between the predicted and**
20 **measured peak capacity is less than 10 %**, as shown in Fig. 8.

21 Fig. 9 shows the comparison of the simulated and measured horizontal displacement responses at
22 exterior joints. In general, the FE models could predict the horizontal movements of the joints well

1 including the inward and outward movements and transition phase. In CSCM, the contour plot of
2 effective plastic strain could indirectly reflect the crack pattern of the specimens as the crack pattern
3 could not be physically displayed in LS-DYNA [24]. Figs. 10 and 11 compare the failure modes of test
4 specimens from numerical simulations and experimental results. In **general**, the FE models could
5 simulate the failure modes and crack patterns well including the positions of rebar fracture and
6 concrete spalling. Therefore, the validated FE models were utilized to further study the effects of
7 boundary conditions, locations of column missing, and story numbers on the varying load transfer
8 mechanism in floors.

9 **4. Detailed Discussion of the Numerical Results**

10 *4.1 Load transfer mechanisms of planar multi-story RC frames*

11 As mentioned above, most of **the** existing tests on progressive collapse research are single-story
12 beam-column substructures or sub-assemblages due to cost and time consideration. However,
13 progressive collapse is a global behavior for a multi-story building, and the load transfer mechanisms
14 may not be the same in **each** story, especially for the asymmetric structure. Therefore, it is necessary to
15 investigate the various load transfer mechanisms of each floor of the frame model. However, the
16 horizontal constraints of the test specimens were simplified due to the limitation of the cast and testing
17 space. Therefore, to get a more realistic response of structures, a five-span planar frame model with
18 penultimate column loss named BFS-P was built based on the verified **modeling** techniques in BFS, as
19 shown in Fig. 12. Comparing to BFS, BFS-P has a close-to-reality boundary condition provided by the
20 beams in surrounding bay. Therefore, BFS-P could be the key reference model in this numerical
21 simulation program. To understand the effect of story numbers on the load transfer mechanism of
22 planar RC beam-column substructures subjected to progressive collapse, BFS-P-5F with five stories,

1 BFS-P-7F with seven stories, and BFS-P-9F with nine stories were also modeled based on the model
2 BFS-P, as shown in Fig. 13.

3 *4.2 Structural resistance of each story*

4 The structural resistance of each story equals the summation of vertical loads on both sides of the
5 beam section located above the removed column. Fig. 14 illustrates the load resistance of each story in
6 specimen BFS-P. As shown in the figure, the load resistance of each story is different after elastic
7 stage. It can be seen that the first story contributed **the** greatest load resistance when the RCD is less
8 than 133 mm or larger than 220 mm. In terms of CAA capacity, the resistance from the third story is
9 larger than that of the second story and **the biggest one is measured in the first story. Regarding the**
10 **TCA stage, the biggest TCA capacity is also measured in the first story.** In general, the assumption of
11 each story demonstrating the same load transfer mechanisms and resistance, which is the basic
12 assumption to use the behavior of a single-story substructure to represent a real multi-story frame, is
13 not accurate for planar frames subjected to a penultimate column missing scenario.

14 Figs. 15, 16, and 17 show the story resistance results of BFS-P-5F, BFS-P-7F, and BFS-P-9F,
15 respectively. Similar to BFS-P, the story resistance began diverging after the elastic stage, and the
16 resistance of the first story is larger than the ones of other stories when the RCD is less than 133 mm
17 or greater than 216 mm. **Besides, prior to the fracture of longitudinal reinforcements, the load**
18 **resistance of the middle stories is quite similar**, indicating that the middle stories have similar load
19 transfer mechanisms. To reveal the behavior of multi-story frame subjected to the loss of a column
20 scenario, commonly utilized single-story sub-assembly tests may be insufficient. **To the contrary,**
21 **three sub-assembly tests (top story, one of middle story, and ground story) with proper boundary**
22 **conditions were required.**

1 *4.3 Development of axial forces in beams*

2 To reveal the difference of load transfer mechanism of the beams in each story, the results of the
3 beam axial force of BFS-P were also extracted and presented in Fig. 18. As the axial force throughout
4 each beam is identical, the development of the axial force of the whole beam can be represented by
5 that of one arbitrary section at the beam. Due to asymmetry, the axial force of the beams at different
6 sides of the removed column may be different. Therefore, the axial forces of the beam sections, which
7 are at a distance of 200 mm away from the beam-column interface, were extracted. The labels of L1 to
8 L3 represent the sections at the left side of the removed column (called interior bay) while the labels of
9 R1 to R3 represent the sections at the right side of the removed column (called exterior bay).

10 In elastic stage, the beams in the first and second stories are in tension while the beam in third
11 story is in compression. These beams worked like a large composite beam under flexure. After elastic
12 stage, as shown in Fig. 18a, the interior-bay beam in the first story (IB-beam-1st) begins to develop
13 compressive force initially and achieved the maximum compressive force of -17.0 kN at a RCD of 108
14 mm. After that, the axial force of the IB-beam-1st starts to decrease, and changes into tension at a RCD
15 of 208 mm. Different from the IB-beam-1st, the axial force of the interior-bay beam in the second story
16 (IB-beam-2nd) is in tension initially, and it transfers to compression at a RCD of 74 mm. The
17 maximum compressive force of the IB-beam-2nd is -4.0 kN, which is only 23.5 % of the one of the IB-
18 beam-1st. Moreover, the axial force of the IB-beam-2nd transfers to tension again at a RCD of 308 mm.
19 For the interior-bay beam in the third story (IB-beam-3rd), the beam is in compression until the RCD
20 reaches 266 mm, and the maximum compressive force is -10.0 kN, which is 58.8 % of the one of the
21 IB-beam-1st. Besides, the maximum tensile forces of IB-beam-1st, IB-beam-2st, and IB-beam-3st are
22 27.0 kN, 11.9 kN, and 8.4 kN, respectively. For the exterior bay, as shown in Fig. 18b, the

1 development of the axial force of the exterior-bay beam in the first story (EB-beam-1st) is quite similar
2 to the one of the IB-beam-1st, which is in compression first and finally in tension. Due to interaction of
3 the beam-column elements among stories, the exterior-bay beam in the second story (EB-beam-2st) is
4 in tension first and in compression slight after RCD of 250 mm. However, the exterior-bay beam in
5 third story (EB-beam-3rd) is always in compression during the whole loading history. In a word, the
6 distributions of axial forces in both interior-bay and exterior-bay beams indicate the CAA could
7 develop in the first and third stories, whereas flexural action is the main mechanism of second story to
8 redistribute the gravity load. Moreover, the significant axial tensile forces of the IB-beam-1st and EB-
9 beam-1st in the large deformation stage also illustrate the TCA could develop in the first story beams
10 effectively.

11 Figs. 19, 20, and 21 show the development of the axial forces in the beams of different stories for
12 BFS-P-5F, BFS-P-7F, and BFS-P-9F, respectively. It is observed that the beam axial forces in the
13 middle stories is quite similar. Most of the beam axial forces in the middle stories are mainly in tension
14 first, and the compressive force appears at large deflection stage, indicating that flexural action is the
15 main mechanism of these stories to balance the gravity load. On the other hand, the beam axial forces
16 of the top and bottom stories are similar to the ones of BFS-P. Similarly, the greatest compressive and
17 tensile forces are measured in the first story, which indicates CAA and TCA can develop in the first
18 story effectively. Further parametric study in Section 5.2 will evaluate the accuracy of the conclusion
19 for the frames subjected to interior or corner removal scenarios.

1 **5. Parametric Study on Planar Multi-Story RC Frame**

2 **5.1 Effect of boundary conditions**

3 As shown in Fig. 2, in the referenced tests [19], the horizontal constraints of the beams in
4 surrounding bay were simplified due to the limitation of the cast and testing space. However, the
5 reliability of the simplification has not been evaluated properly. To quantify the effect of the horizontal
6 restraint stiffness provided by the surrounding bay, four different horizontal restraint stiffness,
7 including 0, 15 kN/mm, 150 kN/mm and rigid, were used for the models of BFS and BFL.

8 Fig. 22 shows the load-displacement curves of BFS and BFL with different boundary conditions.
9 It should be noted that the results of tests are similar to that with rigid restraints for both Specimens
10 BFS and BFL. As shown in Fig. 22, when the horizontal restraint stiffness decreases from rigid to 0,
11 the first peak load (FPL) of BFS and BFL decreases to 87 % and 90 %, respectively, due to weakened
12 CAA. However, reducing the horizontal restraint stiffness is not sensitive to the structural resistance at
13 large deflection stage. Even though there are no spring restraints applied, both BFS and BFL can
14 develop TCA in the initial stage. This is because the remaining two side columns can provide
15 sufficient lateral stiffness to develop TCA initially. However, the TCA weakens due to damage of the
16 side columns later.

17 As shown in Fig. 22a, the FPL of BFS-P, which has a more real boundary condition, is 92 % of
18 that of the BFS. It means that the horizontal restraint stiffness used in the tests may be larger than the
19 real one.

1 **5.2 Effect of location of removed column**

2 For the referenced tests [19], only the scenario of missing a penultimate column is investigated.
3 Two extra numerical models, which were called BFS-I (an interior column was removed in advance)
4 and BFS-C (a corner column was removed in advance), were built to investigate the effects of different
5 column removal scenarios on the load transfer mechanism of each story, as shown in Fig. 23. Fig. 24
6 shows the decomposition of the load resistance of BFS-I. Similar to BFS-P, the first story achieves the
7 highest initial stiffness and provides the majority of CAA and TCA capacity. However, different from
8 BFS-P, the resistances of the second and third stories are almost the same before RCD reached 285
9 mm. The difference in the load resistance of these two stories is mainly due to the mobilization of
10 TCA in the second story. As shown in Fig. 25, when RCD exceeds 285 mm, the beams of the second
11 story start to be in tension, indicating the TCA starts to develop in second story too.

12 For BFS-C, as shown in Fig. 26, the FPL of the first story is also the largest among stories.
13 However, the second story achieves the second largest one, which is different to BFS-P and BFS-I.
14 The different resistance mechanism is due to interaction of the beam-column elements among stories
15 (Vierendeel action).

16 **6. Conclusions**

17 Based on the numerical and parametric studies conducted in this study, the following conclusions
18 are drawn:

19 1. Comparing with experimental results, it is found that high fidelity numerical models are able
20 to accurately simulate the global behavior of the planar multi-story RC frame subjected to a
21 penultimate column loss scenario.

22 2. For a planar multi-story RC frame subjected to a penultimate column removal scenario, the

1 load transfer mechanism of **each** story is not identical. However, when increasing the number of stories,
2 it can be found that the load transfer mechanism of the middle stories is almost the same. Therefore,
3 the behavior of a planar multi-story frame should be equivalently investigated by three types of single-
4 story beam-column assemblies (top-story, middle-story, and ground-story) with proper boundary
5 conditions.

6 3. Horizontal restraint stiffness can significantly affect the development of CAA. Reducing the
7 restraint stiffness of the horizontal springs would decrease the FPL of the frames due to the weakening
8 of CAA. When the horizontal restraint stiffness decreases from rigid to 0, the FPL of BFS and BFL
9 decreases by 87 % and 90 %, respectively. However, horizontal restraint stiffness affecting is
10 insensitive to the development of TCA. Even though spring restraint stiffness reduces to 0, the rest of
11 side columns can provide enough constraints to develop TCA partially.

12 4. It is found from the comparison of the load-displacement curves between the specimens BFS-
13 P and BFS that the load capacity of the Specimen BFS-P is relatively less than that of the Specimen
14 BFS. It means that the horizontal constraints applied on the tests may be stronger than the real one,
15 which will overestimate the capacity of the structure to mitigate progressive collapse.

16 5. Numerical analysis on different column removal scenarios indicates that the beams from a
17 planar multi-story RC frame subjected to progressive collapse demonstrate different load resistance.
18 However, the beam in the first story achieves the greatest initial stiffness and load resisting capacity
19 regardless of the location of removed column.

20

21 **Acknowledgments**

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2 and conclusions expressed in this paper do not necessary reflect the view of Natural Science
3 Foundation of China.

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19

20 **Captions of tables**

21 **Table 1-** Model parameters of CSCM after adjustment (Units: N, mm and ms)

22

23 **Captions of figures**

24 **Fig. 1**–Reinforcement layout of the Specimen BFS: (a) Elevation view, (b) Cross section of RC frame

25 Note: Unit in mm, T=Deformed reinforcing bar; R=Plain reinforcing bar

- 1 **Fig. 2**–Test setup and instrumentation
- 2 **Fig. 3**–Numerical model of Specimen BFS
- 3 **Fig. 4**–Comparison of bond-slip relationship between Model Code 2010 and Contact_1D
- 4 **Fig. 5**–Comparisons of different mesh sizes
- 5 **Fig. 6**–Comparisons of different concrete input parameters
- 6 **Fig. 7**–Unconfined uniaxial stress-strain relationship of concrete for 32.1 MPa based on CSCM
- 7 **Fig. 8**–Comparison of the load-displacement between simulation and test: (a) BFS, (b) BFL
- 8 **Fig. 9**– Comparison of the horizontal movement response from numerical and test: (a) BFS, (b) BFL
- 9 **Fig. 10**–Comparisons of damage patterns of BFS: (a) FEM, (b) Test
- 10 **Fig. 11**–Comparisons of damage patterns of BFL: (a) FEM, (b) Test
- 11 **Fig. 12**–Numerical model of BFS-P
- 12 **Fig. 13**–Numerical models of number of different floors: (a) BFS-P-5F, (b) BFS-P-7F, (c) BFS-P-9F
- 13 **Fig. 14**–Load resistance of each story of BFS-P
- 14 **Fig. 15**–Load resistance of each story of BFS-P-5F
- 15 **Fig. 16**–Load resistance of each story of BFS-P-7F
- 16 **Fig. 17**–Load resistance of each story of BFS-P-9F
- 17 **Fig. 18**–Development of beam axial forces of BFS-P: (a) Left side of removed column, (b) Right side
- 18 of removed column
- 19 **Fig. 19**–Development of beam axial forces of BFS-P-5F: (a) Left side of removed column, (b) Right
- 20 side of removed column
- 21 **Fig. 20**–Development of beam axial forces of BFS-P-7F: (a) Left side of removed column, (b) Right
- 22 side of removed column
- 23 **Fig. 21**–Development of beam axial forces of BFS-P-9F: (a) Left side of removed column, (b) Right
- 24 side of removed column
- 25 **Fig. 22**–Comparison of different boundary conditions: (a) BFS, (b) BFL,
- 26 **Fig. 23**–Numerical models of different column loss: (a) BFS-I, (b) BFS-C
- 27 **Fig. 24**–Load resistance of each story of BFS-I
- 28 **Fig. 25**–Development of beam axial forces of BFS-I
- 29 **Fig. 26**–Load resistance of each story of BFS-C

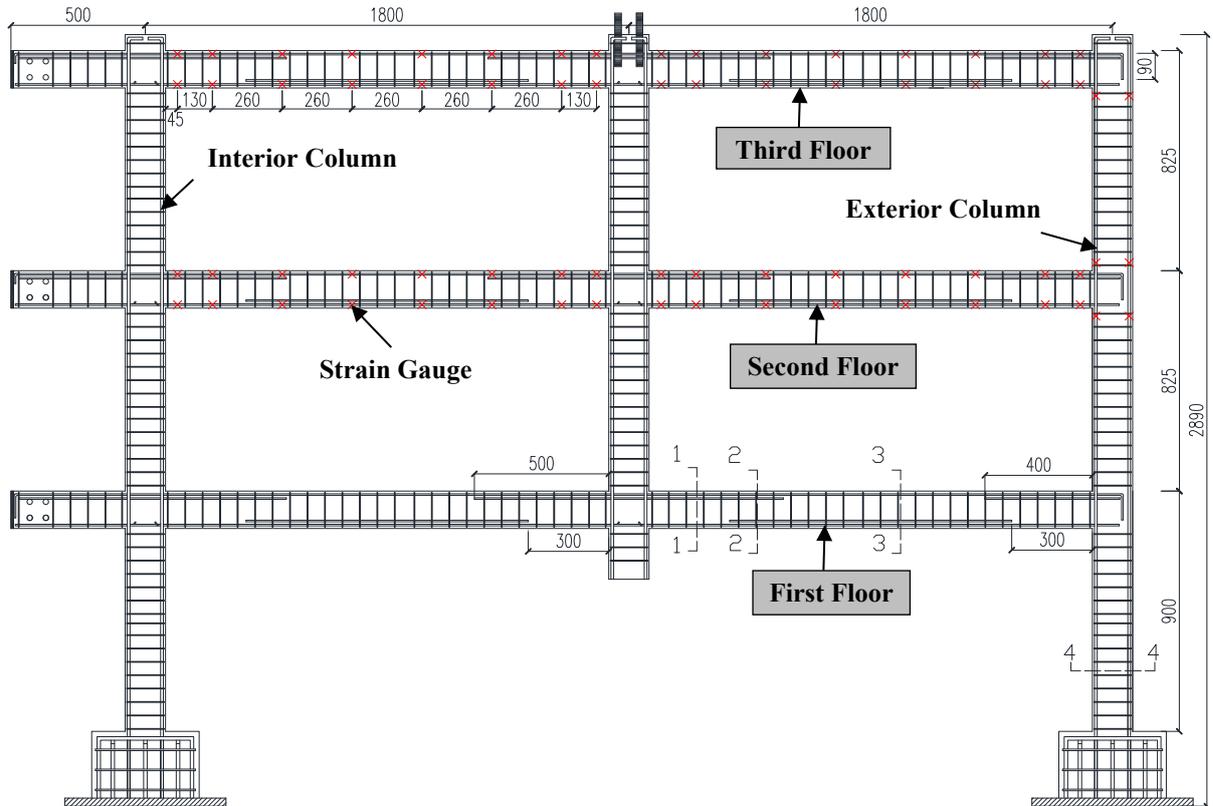
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Table 1
Model Parameters of CSCM after Adjustment (Units: N, mm and ms)

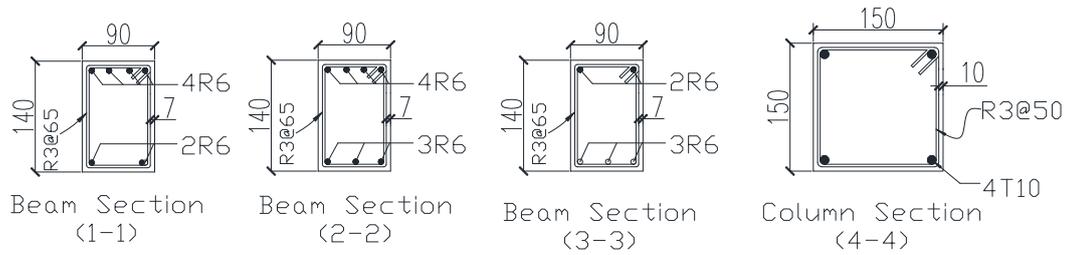
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1	0.00232	1	0.0	0	1.10	0.0	0
PRED							
0							
G	K	ALPHA	THETA	LAMDA	BETA	NH	CH
7065	7738	14.788	0.3029	10.5	0.01929	0	0
ALPHA1	THETA1	LAMDA1	BETA1	ALPHA2	THETA2	LAMDA2	BETA2
0.74735	0.001102	0.17	0.06855	0.66	0.001323	0.16	0.06855
R	XD	W	D1	D2			
5.0	91.5	0.05	2.5e-04	3.492e-07			
B	GFC	D	GFT	GFS	PWRC	PWRT	PMOD
100.0	4.575	0.1	0.04575	0.02288	5.0	1.0	0.0
ETAOC	NC	ETAOT	NT	OVERC	OVERT	SRATE	REPOW
0	0	0	0	0	0	0	0

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(a)

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(b)

Fig. 1–Reinforcement layout of the Specimen BFS: (a) Elevation view, (b) Cross section of RC frame

Note: Unit in mm, T=Deformed reinforcing bar; R=Plain reinforcing bar

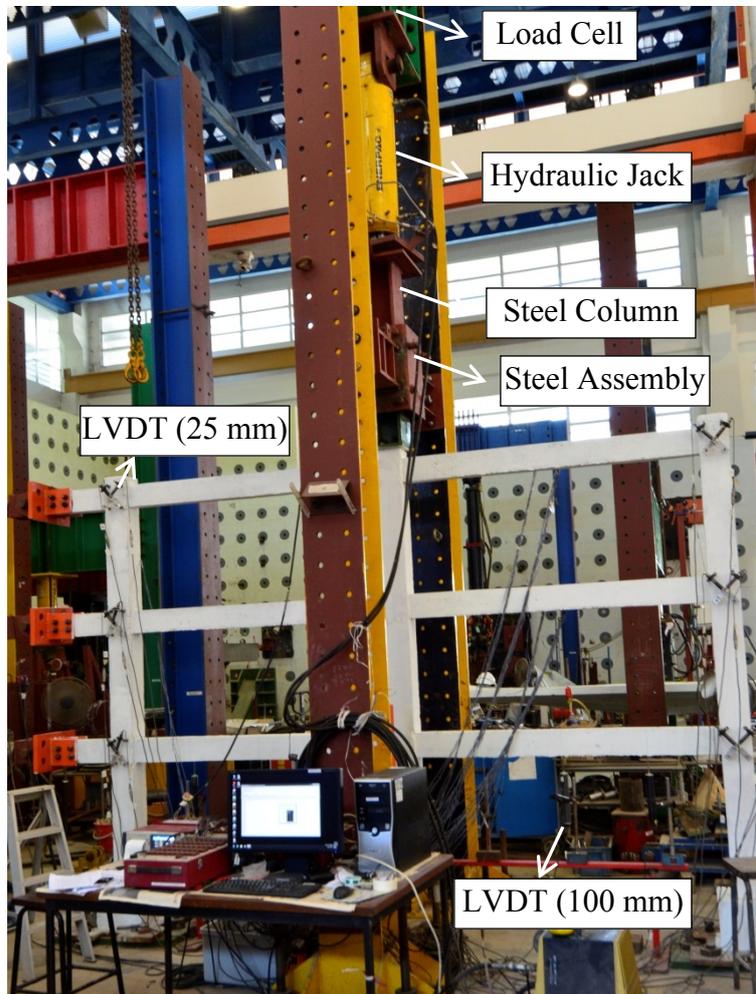
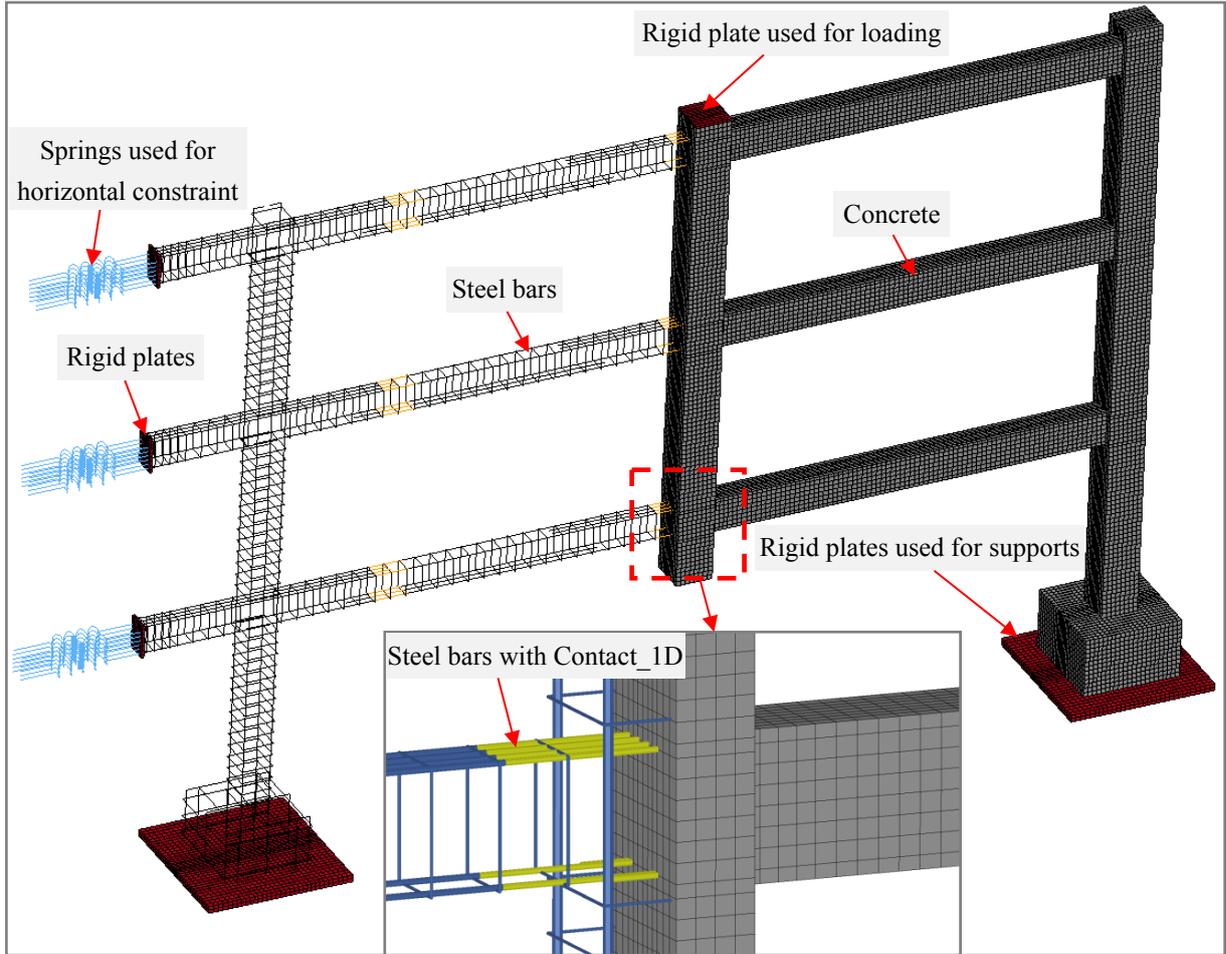


Fig. 2–Test setup and instrumentation

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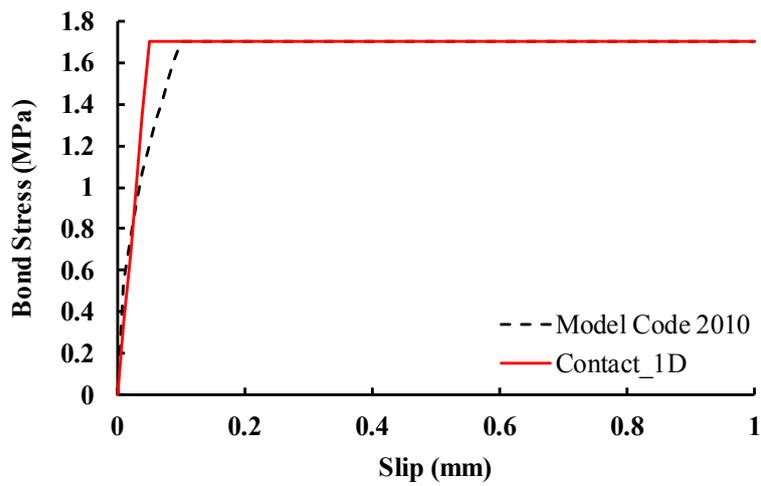
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Fig. 3–Geometric model of Specimen BFS



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Fig. 4–Comparison of bond-slip relationship between Model Code 2010 and Contact_1D

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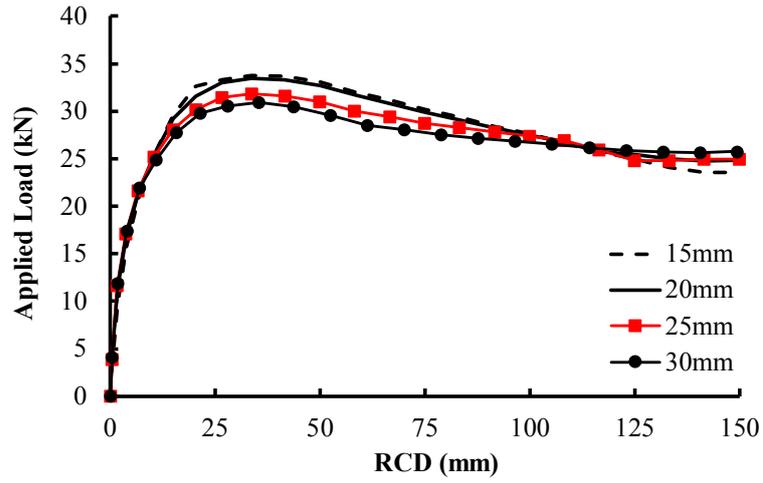


Fig. 5–Effects of different mesh sizes

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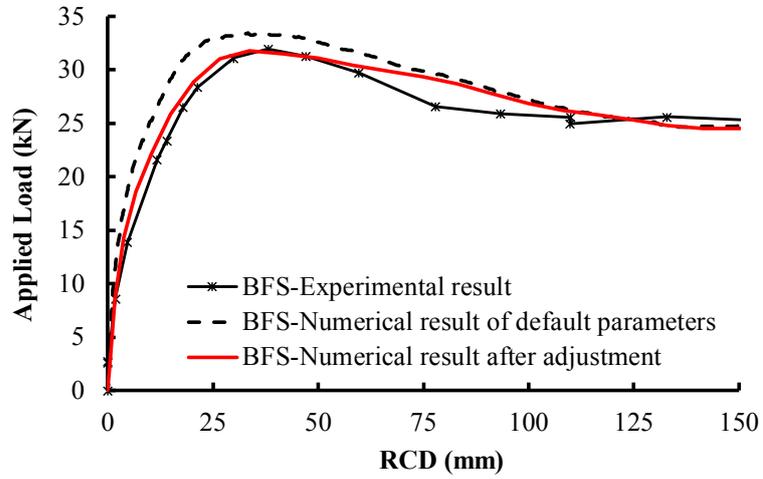


Fig. 6–Effects of different concrete input parameters

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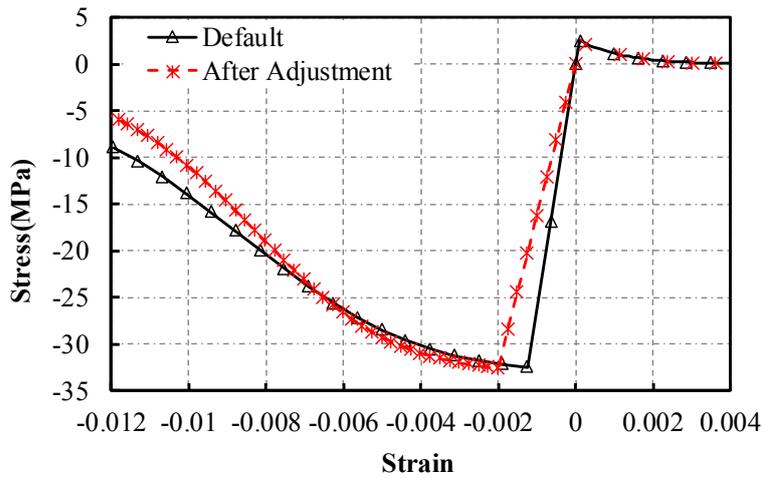


Fig. 7–Unconfined uniaxial stress-strain relationship of concrete for 32.1 MPa based on CSCM

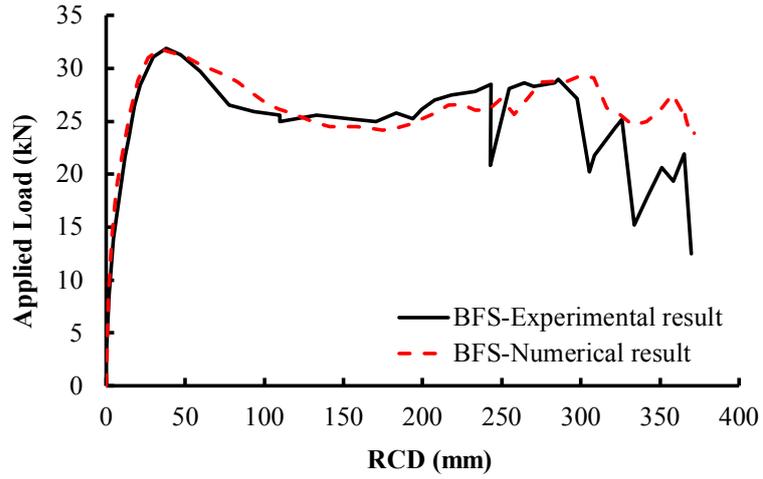
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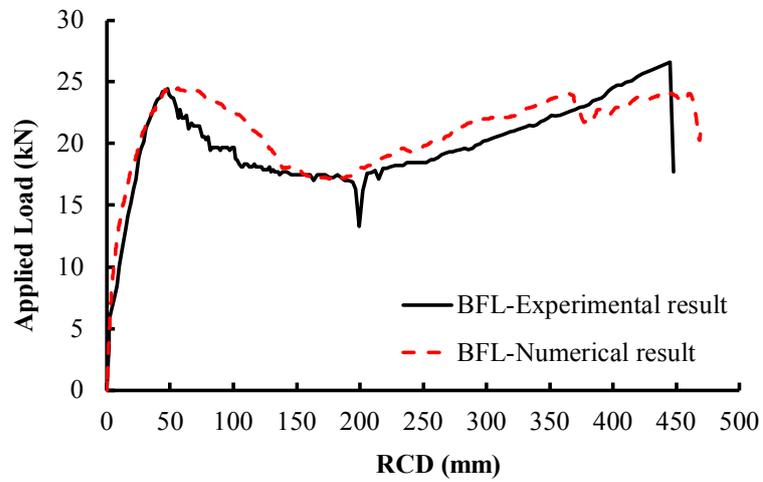
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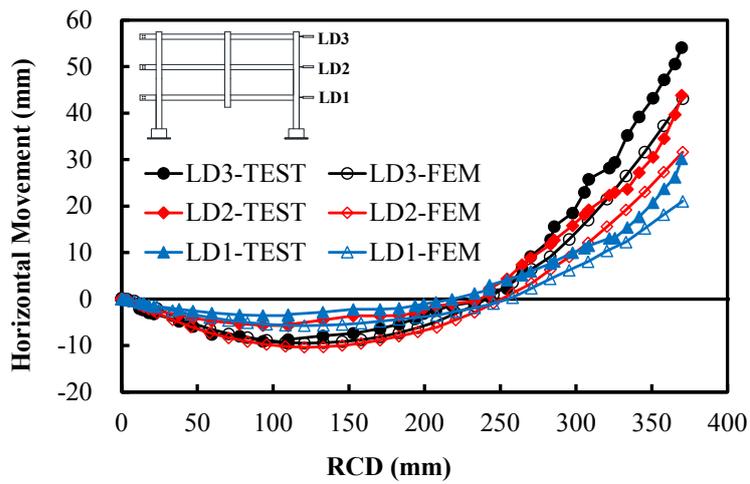


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(b)

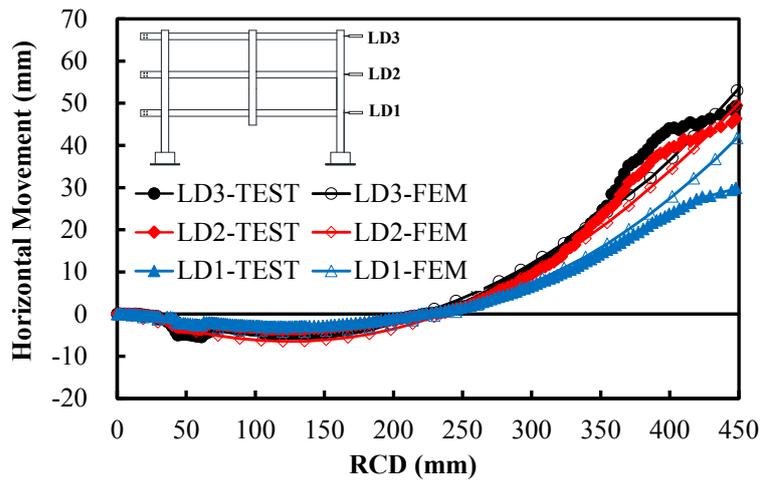
Fig. 8—Comparison of the load-displacement response from numerical and test: (a) BFS, (b) BFL



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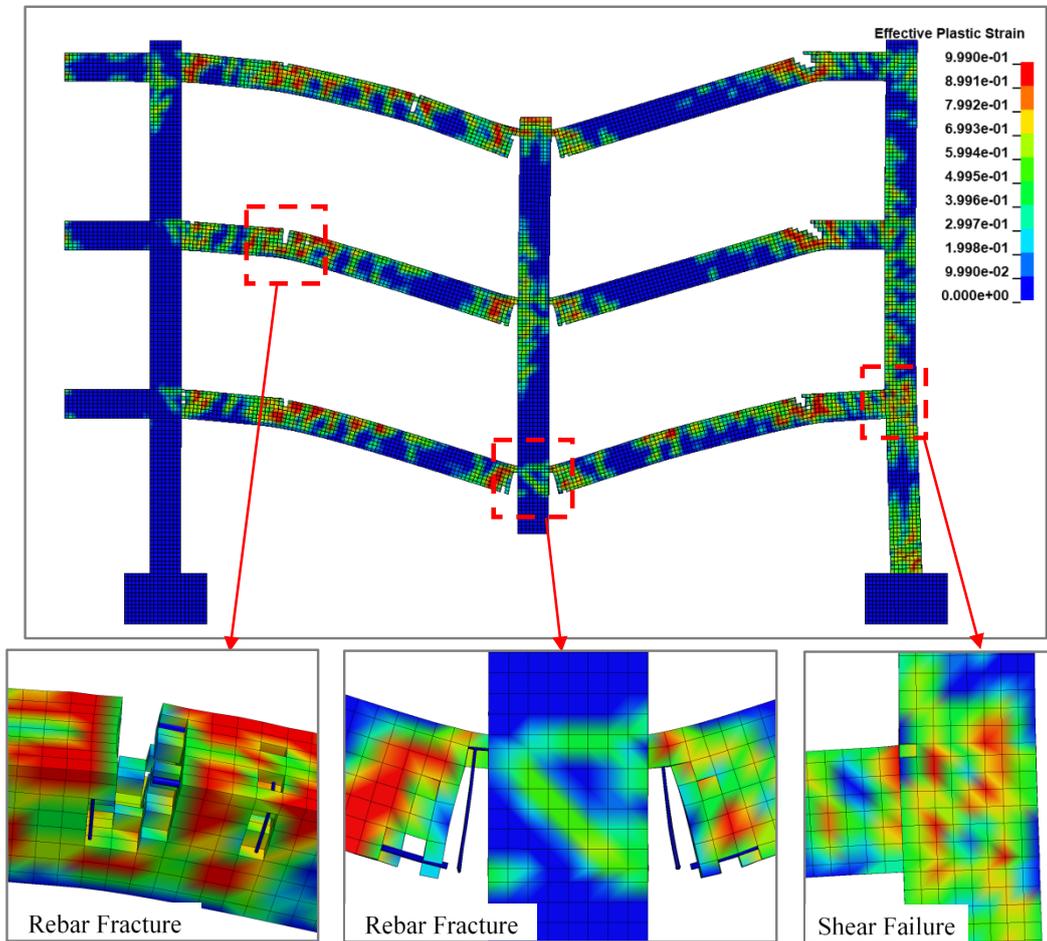
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(b)

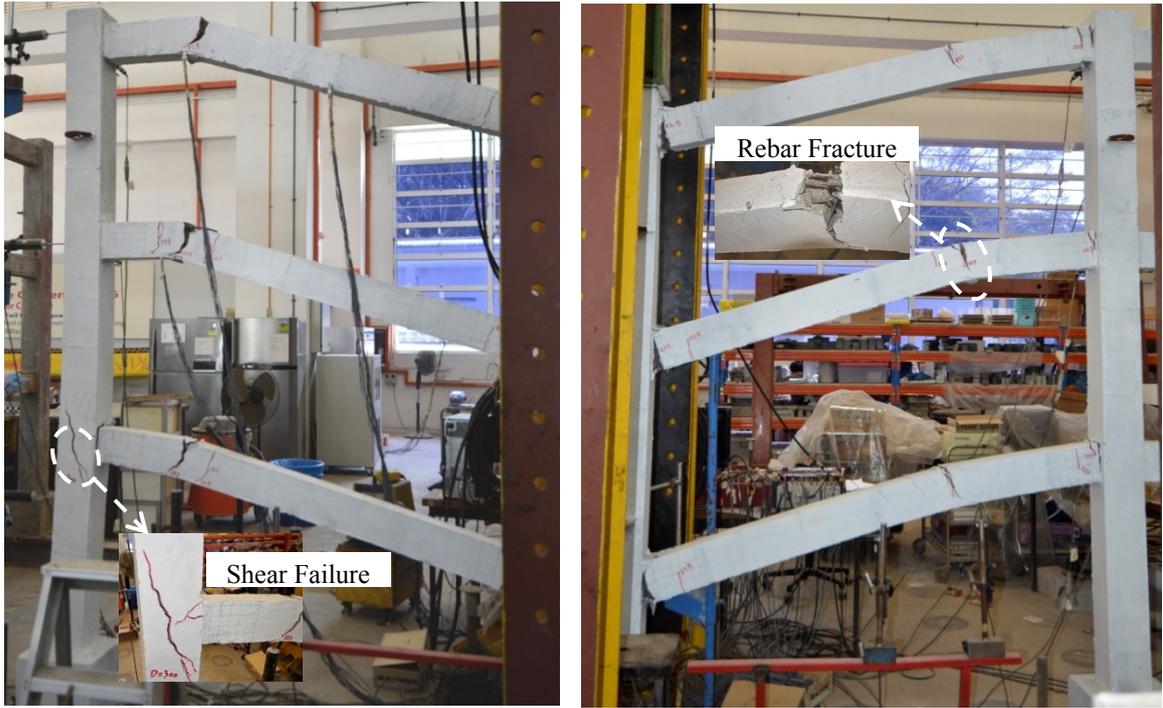
Fig. 9–Comparison of the horizontal movement response from numerical and test: (a) BFS, (b) BFL



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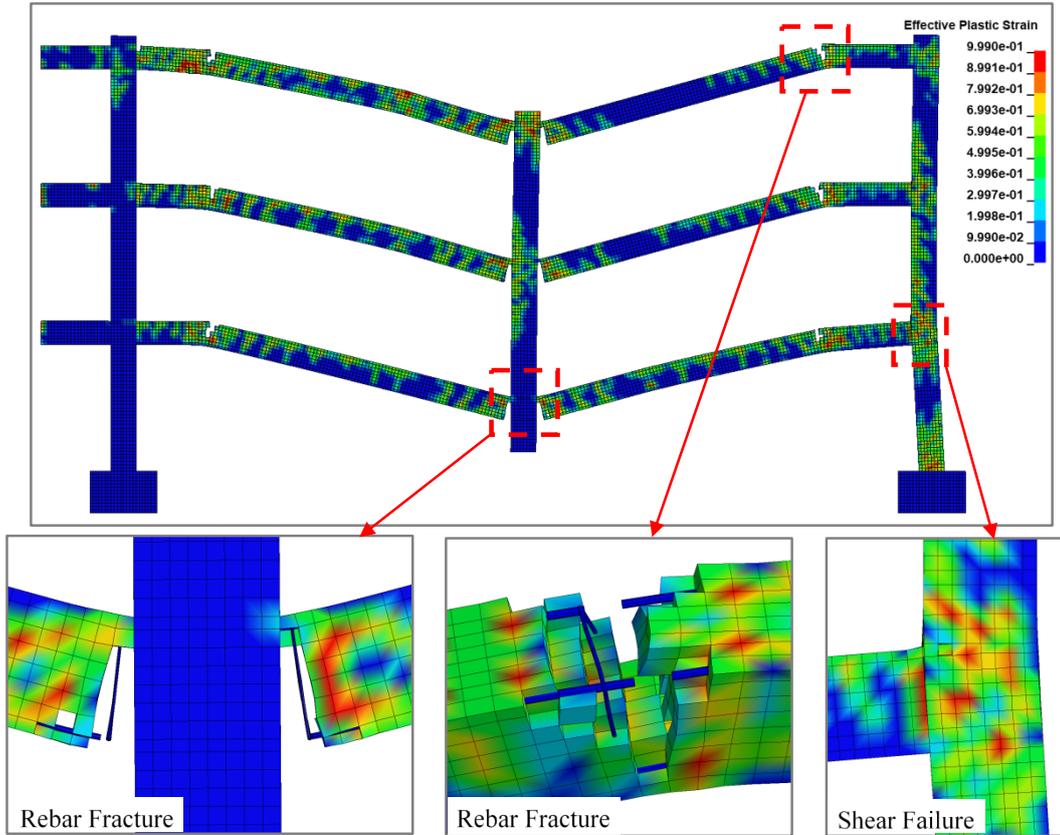
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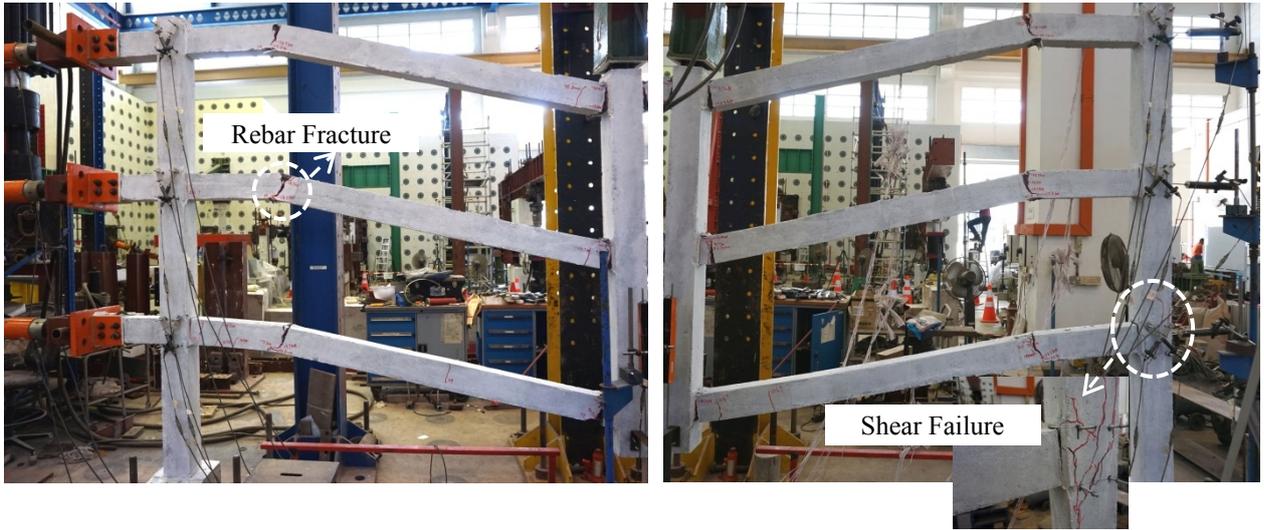
Fig. 10–Comparisons of failure mode of BFS: (a) FEM, (b) Test

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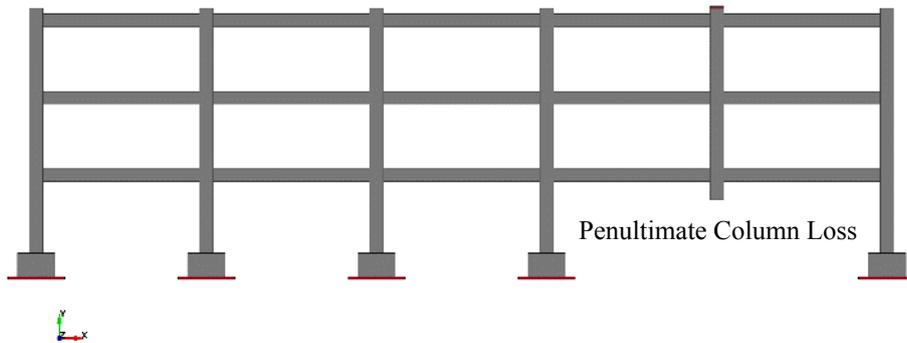
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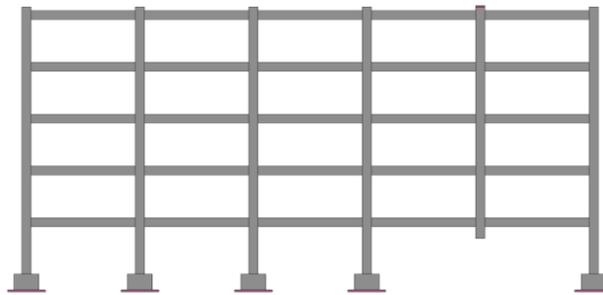
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Fig. 11–Comparisons of failure mode of BFL: (a) FEM, (b) Test



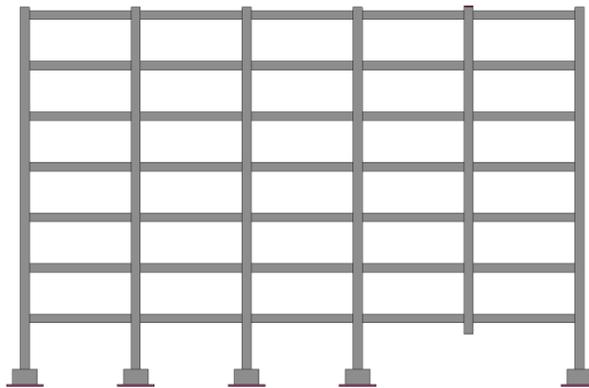
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Fig. 12–Numerical model of BFS-P



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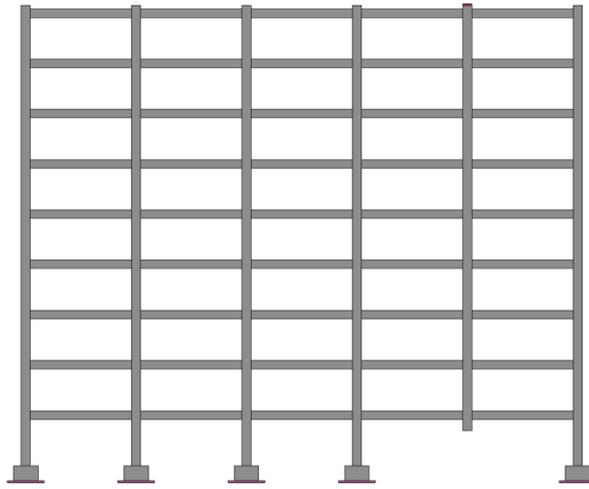
(a)



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(c)

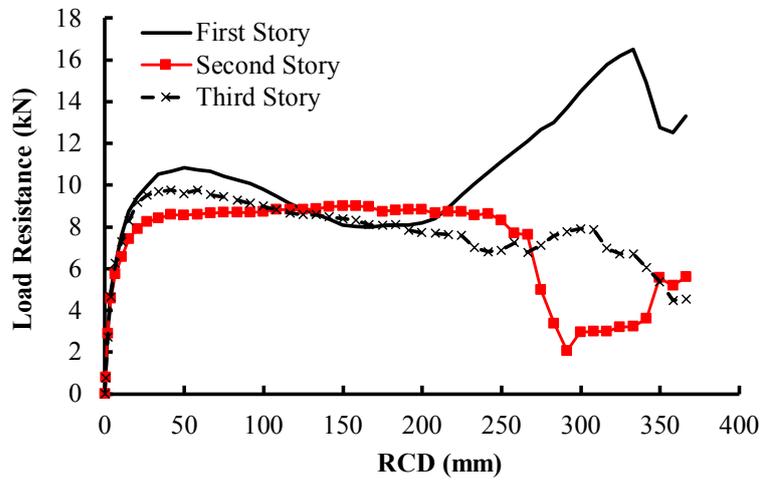
Fig. 13–Numerical models of number of different floors: (a) BFS-P-5F, (b) BFS-P-7F, (c) BFS-P-9F

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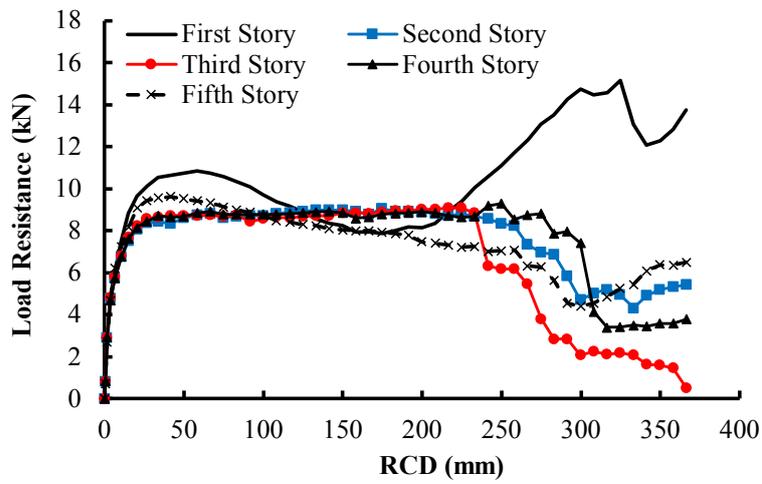


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Fig. 14–Load resistance of each story of BFS-P



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Fig. 15–Load resistance of each story of BFS-P-5F

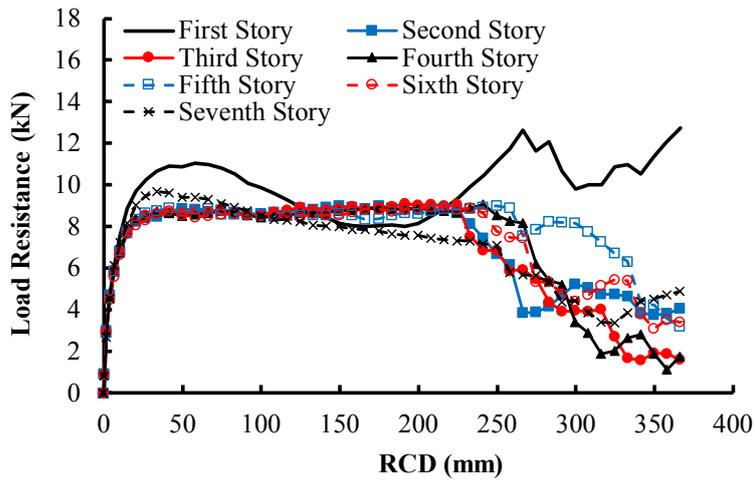


Fig. 16–Load resistance of each story of BFS-P-7F

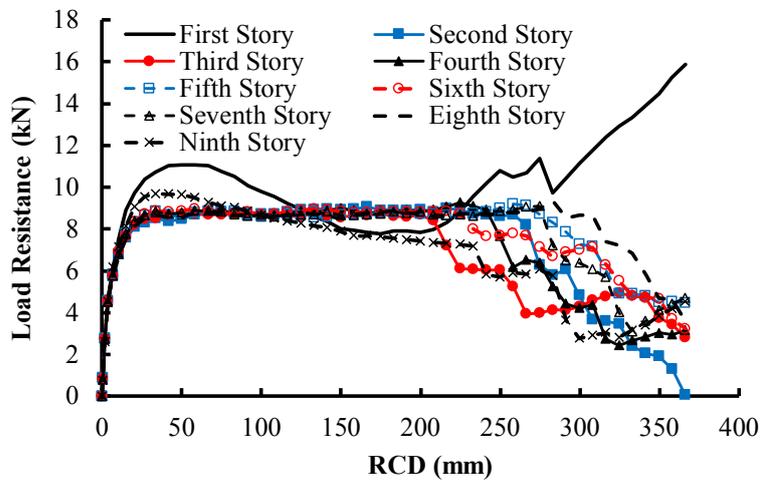
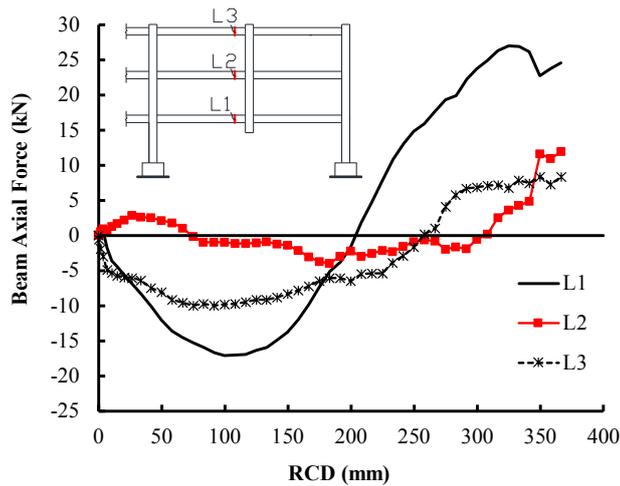
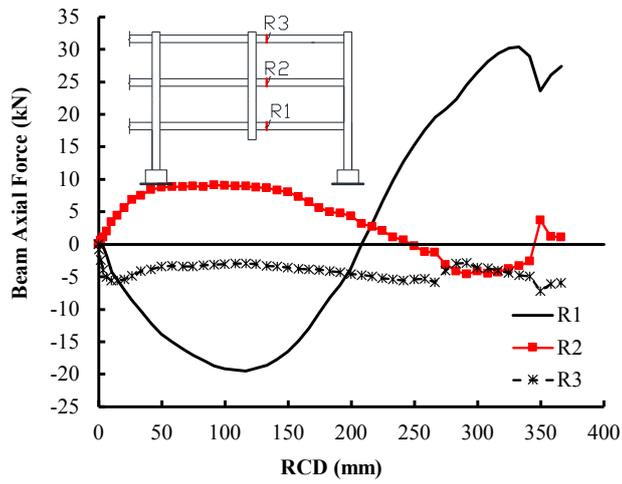


Fig. 17–Load resistance of each story of BFS-P-9F

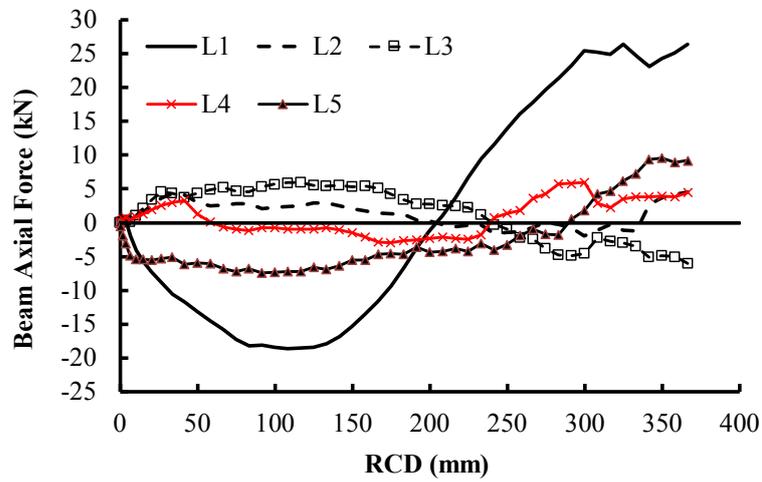


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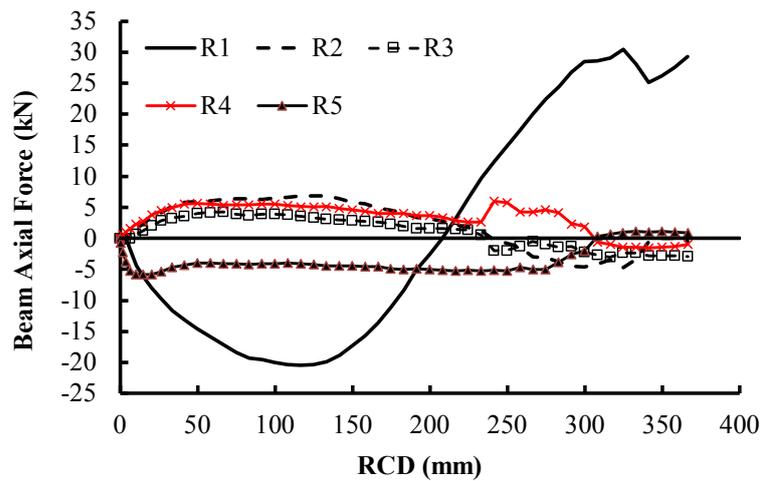


(b)

Fig. 18–Varying of beam axial forces of BFS-P: (a) left side of removed column, (b) right side of removed column

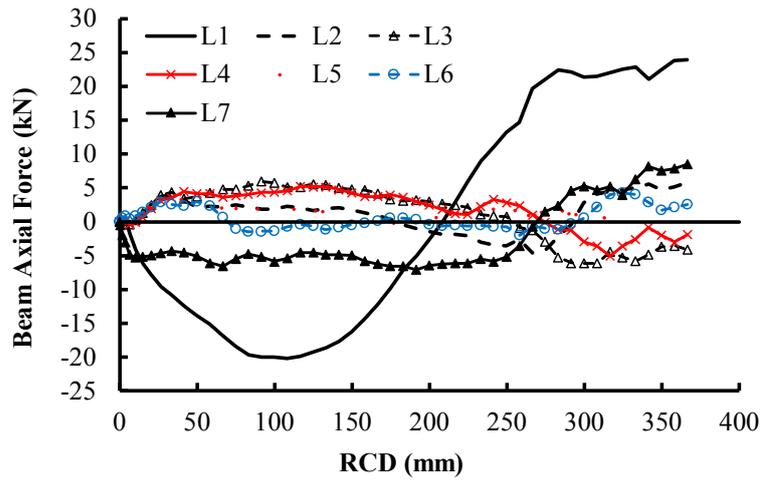


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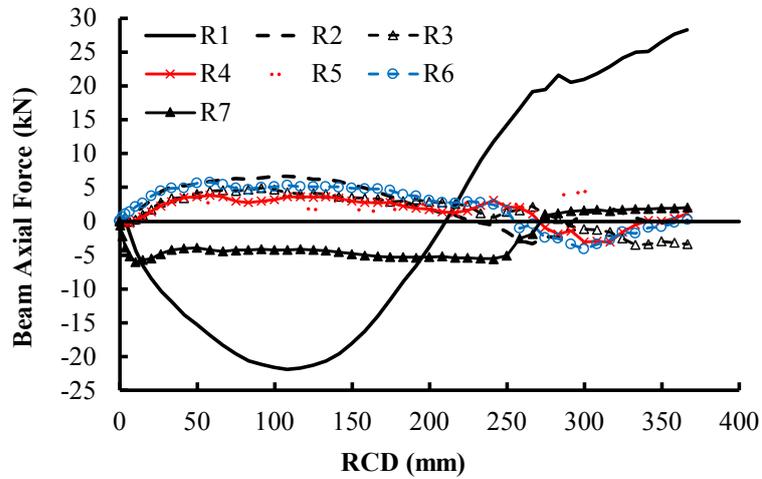


(b)

Fig. 19–Varying of beam axial forces of BFS-P-5F: (a) left side of removed column, (b) right side of removed column

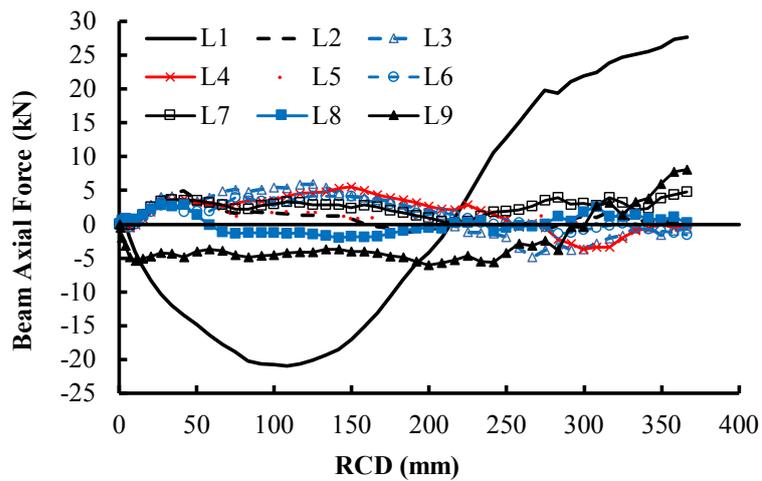


(a)



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Fig. 20—Varying of beam axial forces of BFS-P-7F: (a) left side of removed column, (b) right side of removed column

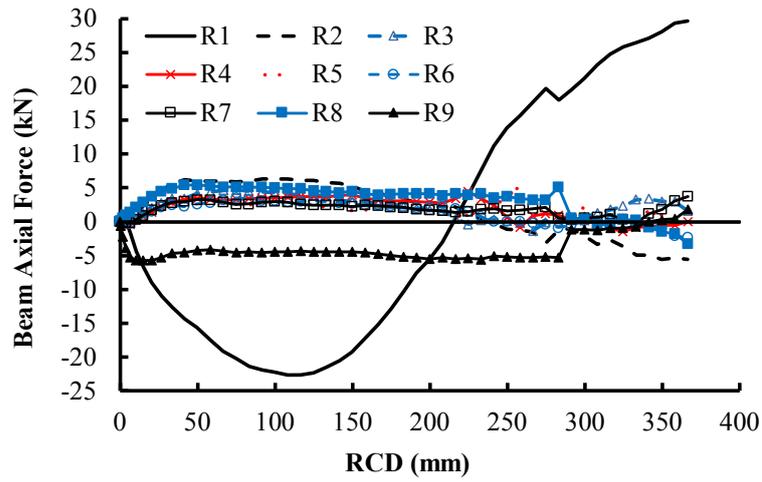


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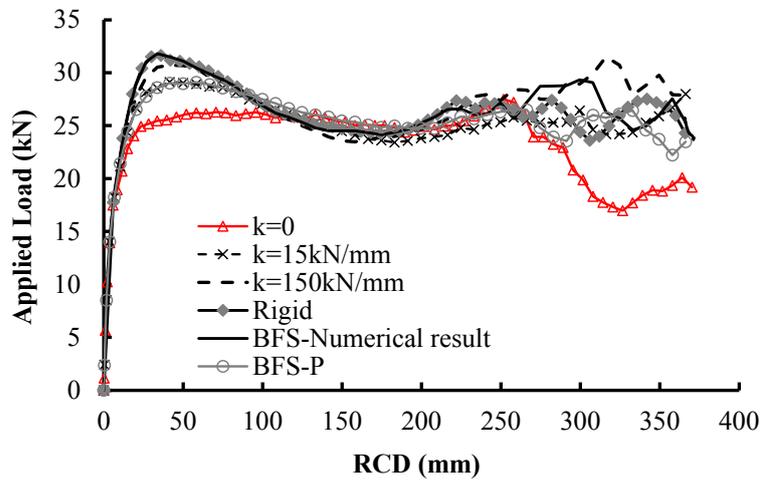
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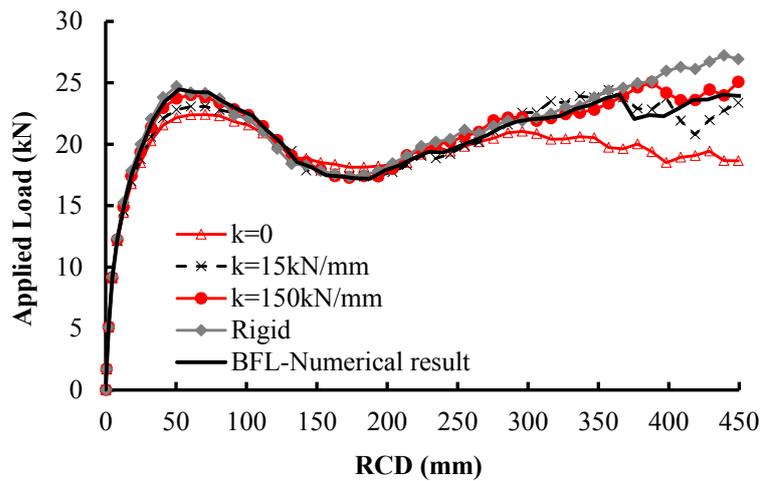


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Fig. 21–Varying of beam axial forces of BFS-P-9F: (a) left side of removed column, (b) right side of removed column

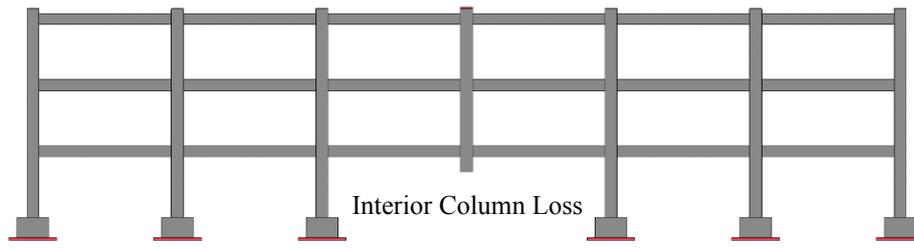


(a)



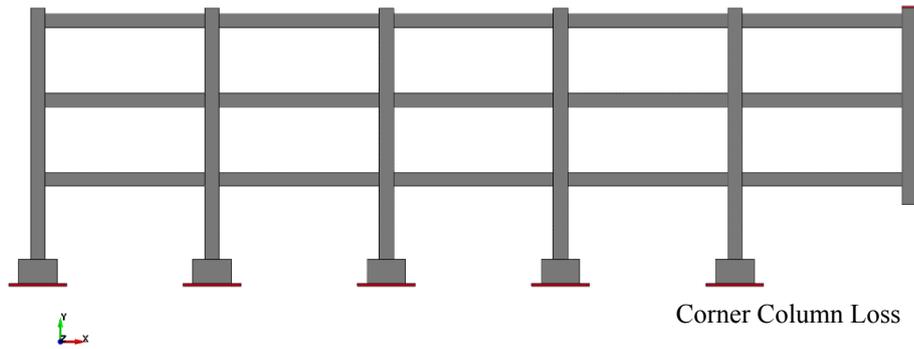
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Fig. 22–Effects of horizontal stiffness of the constraints: (a) BFS, (b) BFL



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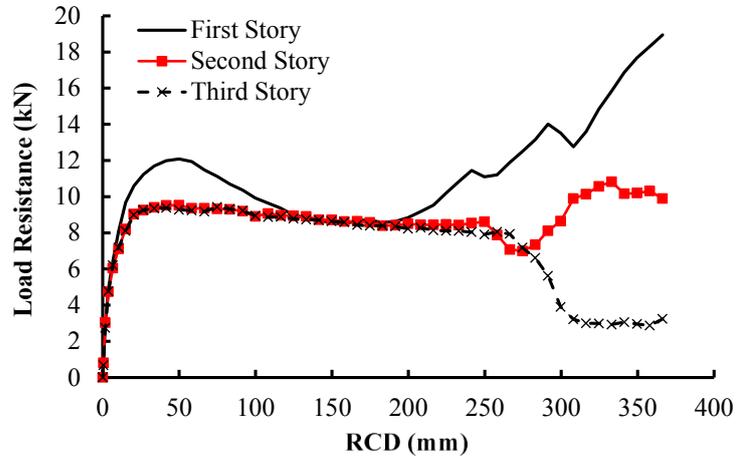


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Fig. 23–Numerical models of different column loss: (a) BFS-I, (b) BFS-C

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Fig. 24–Load resistance of each story of BFS-I

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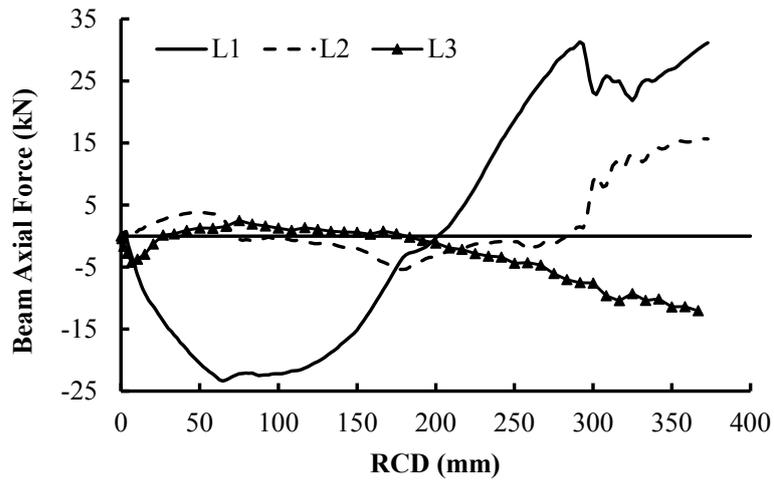


Fig. 25–Varying of axial force of the beams in different story of BFS-I

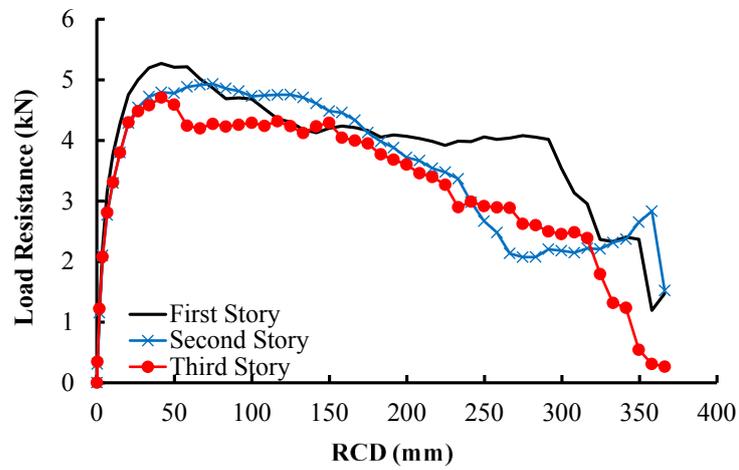


Fig. 26–Load resistance of each story of BFS-C

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Research Highlights

- Multi-storey RC frame was utilized to validate the FE model
- Load resisting mechanisms of each floor in a multi-storey frame is different
- The first storey achieved the largest compressive arch action and catenary action
- The horizontal constraints from surrounding bays is different to rigid constraints

Numerical Evaluation of the Reliability of Using Single-Story Substructures to Study Progressive Collapse Behaviour of Multi-Story RC Frames

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¹College of Civil Engineering and Architecture, Guangxi University, 100 Daxue Road, China, 530004.

² School of Mathematics, Computer Science and Engineering, City, University of London, U.K.

Abstract: Progressive collapse is a global failure for a multi-story building. All stories above the removed column will consequently deform and help redistribute the loads initially withstood by the removed column. However, due to cost and excessive time to be involved, the majority of existing experimental researches regarding progressive collapse rely on single-story beam-column substructures or sub-assemblages. To date, how to use the results from single-story substructures or sub-assemblages to fully or confidently study the behavior of multi-story building is still unclear. Thus, it is imperative to investigate the relationship between the results of single-story substructures and the real behavior of multi-story buildings. Thus, for this purpose, in the present study, a series of planar multi-story reinforced concrete (RC) beam-column substructures were modeled using high-fidelity finite element software LS-DYNA. Firstly, the numerical models were validated by the test results of two three-story planar substructures with different design spans. Secondly, the validated models were explored on various load resistance of each story in the investigated multi-story frame. In addition, the effects of boundary conditions, missing column locations, story numbers on the variation of load resistance were studied in detail using the models.

Keywords: Progressive collapse; Multi-Story RC frames; Load transfer mechanism; Numerical simulations; Column removal scenario

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1. Introduction

Progressive collapse is defined in ASCE/SEI 7 [1] as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it”. To date, there are two main methods to design buildings to mitigate progressive collapse: direct and indirect design methods. For the indirect design method, integrity, redundancy, ductility, and minimum tie-force are required. However, when local damages are triggered, it is difficult to quantitatively evaluate the capacity and behavior of remaining building in resisting progressive collapse based on this method. For the direct design method, alternative load path method is most commonly used as it is event-independent. To understand the behavior of multi-story buildings subjected to sudden column missing scenario, Sasani et al. [2] carried out an on-site test for an actual 10-story reinforced concrete building following the explosive damage of an exterior column. Similarly, a six-story RC infilled-frame building was evaluated following the removal of two adjacent exterior columns simultaneously by Sasani [3]. Song et al. [4] tested a steel frame building subjected to physically removal of four ground columns from one of the perimeter frames to study the load redistribution of the building after each column removal. However, as the service load (live load and partial dead load) was removed prior to on-situ tests, the measured displacement response was little. Majority of on-situ tests only experienced elastic response. The plastic behavior especially for compressive arch action (CAA) and tensile catenary action (TCA) could not be captured and therefore be evaluated in detail. Thus, the majority of existing tests in laboratory regarding progressive collapse were single-story beam-column substructures based on alternative load path method (relied on push-down loading regime). A number of tests [5-13] quantified the effects of geometric characteristic and reinforcing details on the mobilization of CAA and TCA for progressive collapse prevention. It was found that the span/depth ratio has great effects on the mobilization of CAA when the RC frame

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123 1 subjected to the loss of a middle column scenario. In addition, the CAA capacity is sensitive to the
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125 2 horizontal stiffness provided by the beam ends. However, the amount of longitudinal reinforcements in
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127 3 the structural concrete members has little effect on developing CAA. The researchers [8] indicated that
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129 4 the continuous top longitudinal reinforcements contributed to TCA capacity while Yi et al. [14]
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131 5 indicated that both top and bottom longitudinal reinforcements provided contributed to TCA capacity.
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135 6 Moreover, several studies [15-17] were carried out to evaluate the dynamic response and dynamic
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137 7 load increase factor of RC beam-column sub-assemblages subjected to sudden column removal
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139 8 scenario. Qian and Li [15] indicated that the acceleration of the frame after sudden column removal
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141 9 could be as large as 3.5g, where g is the acceleration of gravity, and the dynamic load increase factor
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143 10 could be less than 1.38. Qian and Li [16] quantified the slab effects on the dynamic response of RC
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145 11 frames subjected to the sudden removal of a ground corner column. In addition, they proposed an
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147 12 equivalent single-degree-of-freedom (SDOF) model to predict the dynamic ultimate load capacity of
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149 13 the tested specimens. The dynamic load increase factor of tested specimens was ranged from 1.30 to
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151 14 1.34. Liu et al. [17] investigated the dynamic behavior of steel frames with different connections
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153 15 subjected to sudden removal of a center column experimentally. The test results indicated that the
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155 16 dynamic phenomenon may detriment the behavior of steel connections and degrade the progressive
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157 17 collapse resistance of the substructures.
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164 18 However, the reliability of using single-story substructures to study the behavior of multi-story
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166 19 buildings is based on the assumption that the stories above the removed column have identical
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168 20 performance, which is questionable and has not been proved. Weng et al. [18] used high fidelity finite
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170 21 element (FE) models to investigate the load resisting mechanisms of each story for a multi-story flat
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172 22 slab structure under a middle column loss scenario. The numerical results indicated that the load
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174 23 resistance from each story in a multi-story flat slab building was different and the largest load resisting
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183 1 capacity occurred in the first story. However, for a multi-story RC frame, it may be different to flat
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185 2 slab structures as beams could help to redistribute the loads. In Qian and Li [19], two multi-story
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187 3 frames were tested based on displacement-controlled push-down method. As it is not feasible to test a
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189 4 multi-story frame to assess the load resisting contribution of each story, FE models, which are
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191 5 validated against the experimental tests, are used for deeper understanding of the various load transfer
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193 6 mechanism and load resisting contribution of each floor consequently. Moreover, the effects of the
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195 7 number of stories, boundary conditions, and missing column locations are also studied using the
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197 8 validated numerical models.

201 9 **2. Previous Experimental Work**

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203 10 A quasi-static experimental study on progressive collapse resistance of planar RC beam-column
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205 11 substructures subjected to push-down loading regimes was conducted by Qian and Li [19] and the test
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207 12 results of two bare frames are used to validate the reliability of FE models in this numerical study.
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209 13 These two specimens (BFS and BFL) were one-quarter scaled. They were assumed to be subjected to
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211 14 the loss of a penultimate column. The dimension and reinforcement details of Specimen BFS are
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213 15 shown in Fig. 1. As can be seen in the figure, the beam span was 1800 mm. The story height was 900
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215 16 mm in the first story and 825 mm in upper stories. The cross section of the beam and column was 90
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217 17 mm×140 mm and 150 mm×150 mm, respectively. The concrete clear cover was 7 mm and 10 mm for
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219 18 beam and column, respectively. Enlarged foundation base with a size of 400 mm×300 mm was
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221 19 designed at the toe of side columns for fixing. The hoop stirrups with 90 degrees bends were utilized
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223 20 for transverse reinforcements. As it was non-seismically designed, no transverse reinforcements were
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225 21 placed in the joint region. The curtailment of longitudinal reinforcements in the beam was in
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227 22 accordance with Singapore Code CP-65 [20]. The middle column in ground level is assumed removed
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229 23 before test and thus, the middle column was only fabricated the upper two stories. For Specimen BFL,
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243 1 similar reinforcement details and dimensions to Specimen BFS were used except longer span of 2400
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245 2 mm designed. Average cylinder compressive strength measured on the days of testing for both
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247 3 specimens was 32.1 MPa. The yield strength of R3, R6, and T10 were 417 MPa, 449 MPa, and 515
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249 4 MPa, respectively. The ultimate strength of R3, R6, and T10 were 479 MPa, 537 MPa, and 594 MPa.
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251 5 The measured elongation ratio of R3, R6, and T10 were 9.7 %, 13.3 %, and 16.9 %, respectively. “R”
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253 6 represents plain reinforcement while “T” represents deformed reinforcement.
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257 7 The typical experimental setup and locations of instrumentations are shown in Fig. 2. As
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259 8 shown in the figure, the specimens were fixed to the strong floor by the foundation bases, which were
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261 9 cast monolithically with the side columns. A steel column and a specially designed steel assembly
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263 10 were installed to avoid unforeseen out-of-plane movement of the specimen. This specially designed
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265 11 steel assembly only allows vertical movement of the middle column through constraining its rotational
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267 12 and horizontal movements. A hydraulic jack with a 600 mm stroke was installed on the steel column to
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269 13 apply vertical load. It should be noted that the displacement-controlled push-down loading method was
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271 14 adopted in the reference tests [19]. A load cell was installed above the hydraulic jack to measure the
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273 15 applied vertical load. A roller together with a tension/compression load cell was installed horizontally
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275 16 at each extension part of the specimen to simulate the horizontal constraints of the beams in the
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277 17 surrounding bay.
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283 18 **3. Numerical Model Development and Validation**

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285 19 To illustrate the variation of load resisting capacity and mechanisms of the beams in different
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287 20 stores overtly, explicit solver software LS-DYNA [21] was used to build the FE models due to its
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289 21 numerical stability and various constitutive models available. The FE models were validated based on
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291 22 the experimental results first. As shown in Fig. 3, similar boundary conditions as experimental tests
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293 23 were used at the FE models. As gaps were deliberately left near the horizontal constraints for the
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303 1 facility of installation, springs were installed horizontally at the beam ends of the extension part. The
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305 2 stiffness of the spring was determined by the measured horizontal reaction force and horizontal
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307 3 movements at the ends, which was roughly equal to 80 kN/mm at each beam end with a total of 240
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310 4 kN/mm.
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312 313 314 5 **3.1 Element types** 315

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317 6 Fig. 3 shows the numerical model of Specimen BFS. Concrete is simulated by 8-node solid
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319 7 elements with reduced integration scheme. This solid formulation only has one integration point in
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321 8 each element, which can enhance computational efficiency with the promise of sufficient accuracy, but
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323 9 hourglass control should be defined properly when this type of element was adopted. Moreover,
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325 10 reinforcing bars were modeled using 2-node Hughes-Liu beam elements with 2×2 Gauss quadrature
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327 11 integration at the cross-section. This beam formulation can simulate the behavior of axial force, bi-
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329 12 axial bending, and finite transverse shear strains [22]. Furthermore, the rigid plates for supports or
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331 13 loading points were also modeled by 8-node solid elements and the springs for horizontal restraints
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333 14 were simulated by discrete elements.
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339 340 15 **3.2 Bond-slip relationship simulation** 341 342

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344 16 To improve the accuracy of modeling, considering the bond-slip relationship between
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346 17 reinforcement and concrete is important, as the perfect bonding assumption used in other models will
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348 18 cause over-prediction of load-carrying capacity and lead to premature fracture of reinforcement due to
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350 19 stress concentration [23]. In this numerical study, the bond-slip relationship between reinforcement
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352 20 and concrete material was considered by using keyword *CONTACT_1D to define one-dimensional
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363 1 contact interface between the concrete and rebar elements. Virtual springs are defined between the
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365 2 slave nodes from beam elements and the master nodes from solid elements, and the spring force
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367 3 depends on relative displacements between the slave and master nodes [21]. However, simulating
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369 4 bond-slip behavior for all reinforcements would be complicated and required more computational
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371 5 resources. Based on test results, it was noted that the slip occurred mainly at the beam-column joints
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373 6 and the curtail point of top beam longitudinal reinforcements. As a result, similar to previous studies
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375 7 [24, 25], the CONTACT_1D function was only used for the beam longitudinal reinforcements at the
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377 8 location of beam-column joints and the reinforcements near curtail points, as shown in Fig. 3. Besides,
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379 9 the remaining reinforcements were assumed to be perfect bonding to concrete using keyword
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381 10 *Constrained_Lagrange_In_Solid. To calibrate the properties of Contact_1D, the bond-slip
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383 11 relationship proposed by fib Model Code 2010 [26] was applied. For monotonic loading, the bond
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385 12 stress τ_b between concrete and hot-rolled plain bar for pull-out failure can be calculated as following
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387 13 [26]:

$$\begin{cases} \tau_b = \tau_{b\max} (s / s_1)^{0.5} & \text{for } 0 \leq s \leq s_1 \\ \tau_b = \tau_{b\max} = 0.3\sqrt{f_c} & \text{for } s > s_1 \end{cases} \quad (1)$$

397 15 Where $s_1 = 0.1 \text{ mm}$; f_c is standard cylinder compressive strength. In the tests, the compressive
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399 16 strength of concrete is 32.1 Mpa. Therefore, $\tau_{b\max} = 1.70 \text{ MPa}$.

402 17 For the CONTACT_1D (LS-DYNA) function [21], the bonding relationship between the beam
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404 18 elements and solid elements is assumed to be elastic-perfectly-plastic. After elastic stage, the bond
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406 19 stress would decay following an exponential damage curve. The constitutive law of shear stress τ and
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408 20 slip s is given as [21]:

$$\begin{cases} \tau = G_s s & \text{for } s \leq s_{\max} \\ \tau = \tau_{\max} e^{-h_{\text{dmg}} D} & \text{for } s > s_{\max} \end{cases} \quad (2)$$

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423 1 where G_s is bond shear modulus; s_{\max} is maximum elastic slip; h_{dmg} is damage curve exponential
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425 2 coefficient; D is damage parameter, which is equals $(s-s_{\max})$.
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427 3 The comparison between Eqs. (1) and (2) indicates that the values of h_{dmg} and τ_{\max} are equal to 0
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430 4 and 1.7 Mpa, respectively. Based on the suggestion from Pham et al. [24] and Yu et al. [25], $s_{\max} = 0.5$
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432 5 $s_l = 0.05$ mm. Therefore, $G_s = \tau_{\max} / s_{\max} = 34 \text{MPa} / \text{mm}$. The comparison of the bond-slip relationship
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435 6 between the fib Model Code 2010 [26] and the suitable model used in CONTACT_1D is shown in Fig.
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441 8 **3.3 Material model**

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445 9 In this study, Continuous surface cap model (CSCM) is chosen to simulate concrete material. This
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447 10 model can effectively model damage-based softening and modulus reduction, shear dilation, shear
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450 11 compaction, confinement effect, and strain rate effect under low constraint conditions [27]. Previous
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452 12 studies had proven its accuracy in the simulation of RC components under both quasi-static and
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454 13 dynamic conditions [24, 25, 28].
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457 14 The CSCM provides a simplified version (*Mat_CSCM_CONCRETE) for concrete materials
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459 15 with the compressive strength between 28 Mpa and 48 Mpa. The default parameters depend on three
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462 16 input parameters: unconfined compressive strength f'_c , maximum aggregate size A_g , and units. For both
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464 17 Specimens BFS and BFL, f'_c and A_g are 32.1 Mpa and 8 mm, respectively. The CSCM also provides a
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466 18 strain-based approach of erosion algorithm to simulate material failure, and the related parameter is
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469 19 “ERODE”. When the “ERODE” is set greater than 1.0, the concrete elements would be deleted if
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471 20 damage index exceeds 0.99 and the maximum principal strain exceeds (ERODE-1.0) according to LS-
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474 21 DYNA keyword user’s manual [21]. This feature is used here to effectively model the failure mode of
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476 22 the frame. In the reference test results, the failure modes of the multi-story frames were governed by
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483 1 the flexure and tensile actions, primarily denoted by the formation of severe cracks at the beam ends
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485 2 near the center column and at the curtail points of beam top longitudinal reinforcements. Therefore, the
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487 3 maximum principal strain is a suitable criterion for erosion algorithm. The value of “ERODE” is mesh-
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490 4 dependent, and set as 1.10, corresponding to the maximum principal of 0.1, for element size 20 mm
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492 5 according to the previous work [24]. The strain rate effect of the CSCM is ignored because only quasi-
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495 6 static behavior is considered.

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497 7 The isotropic elastic-plastic material model Mat_Plastic_Kinematic (MAT_003) is used to model
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499 8 reinforcements. The elastic modulus, yield strength, tangential modulus after yielding, and ultimate
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502 9 strain is determined based on properties of steel bars. Also, the strain rate effect is excluded.

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504 10 As choosing the appropriate mesh size is important to obtain reliable and effective results, mesh
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507 11 sensitivity is evaluated. Four different mesh sizes of elements (side length for solid elements and
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509 12 length for beam elements), including 30 mm, 25 mm, 20 mm, and 15 mm, were employed for
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511 13 Specimen BFS. The results of the load-removed column displacement (RCD) relationship for different
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514 14 mesh sizes are shown in Fig. 5. Obviously, mesh size of 20 mm is adequate, as further mesh
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516 15 refinement is not able to cause any remarkable convergence but instead taking larger computational
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519 16 resources. As a result, the mesh size is chosen as 20 mm.

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521 17 However, based on the default parameters of the CSCM, the numerical models will overestimate
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523 18 the initial stiffness and load resisting capacity of the specimens, as shown in Fig. 6. Unconfined
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526 19 uniaxial stress-strain relationship for 32.1 MPa based on the default parameters of the CSCM is shown
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528 20 in Fig. 7. As can be seen from the figure, the compressive strength is attained at a strain of 0.001. But
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531 21 in reality, when normal strength concrete reaches its compressive strength, the strain is usually at about
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533 22 0.002. Therefore, the elastic modulus of concrete should be reduced properly to improve the numerical
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536 23 results, which had been pointed out by Yu et al. [25]. On the other hand, Yu et al. [25, 28] suggested

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543 1 that the tensile fracture energy G_{ft} could be reduced to 80 % of the default one when the simulating
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545 2 result is over predicted. If shear or compressive based damage is significant, then setting G_{fs}
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547 3 (compressive fracture energy) = $0.5 G_{ft}$ and G_{fc} (shear fracture energy) = $50 G_{ft}$ is reasonable [25].
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550 4 However, the default ones are assumed as $G_{fs}=G_{ft}$ and $G_{fc}=100G_{ft}$. Since severe shear cracks were
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552 5 formed in the exterior joint in the first floor and the concrete crushing was not obvious during testing,
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555 6 only the reduced shear fracture energy is used for the CSCM model herein. The user-specified material
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557 7 property inputs for CSCM are listed in Table 1. When the adjusted material property is used for
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559 8 simulating, the stiffness of the unconfined uniaxial compression is lower than that of the default one
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562 9 and the compressive stress reduces faster in the softening stage, as shown in Fig. 7. However, the
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565 10 adjusted material property can improve the numerical results significantly, as shown in Fig. 6.
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567 11 Therefore, these adjustments are finally used to simulate Specimens BFS and BFL.

570 12 **3.4 Verification of numerical model**

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574 13 Fig. 8 shows the comparison of load-displacement curves from numerical simulation and
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577 14 experimental results. Generally, the FE models can simulate all three stages of the structural responses
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579 15 well. In the first stage, the structural resistance increases until reaching the first peak load. The
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582 16 resistance is attributed into the flexural action and compressive arch action (CAA). Then, the
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584 17 resistance decreases due to the weakening of CAA in the second stage. In the last stage, the resistance
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587 18 increases again due to the development of tensile catenary action (TCA), and abrupt reduction since
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589 19 rebar fracture was also simulated. For Specimens BFS and BFL, the error between the predicted and
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591 20 measured peak capacity is less than 10 %, as shown in Fig. 8.

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594 21 Fig. 9 shows the comparison of the simulated and measured horizontal displacement responses at
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596 22 exterior joints. In general, the FE models could predict the horizontal movements of the joints well

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603 1 including the inward and outward movements and transition phase. In CSCM, the contour plot of
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605 2 effective plastic strain could indirectly reflect the crack pattern of the specimens as the crack pattern
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607 3 could not be physically displayed in LS-DYNA [24]. Figs. 10 and 11 compare the failure modes of test
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609 4 specimens from numerical simulations and experimental results. In general, the FE models could
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611 5 simulate the failure modes and crack patterns well including the positions of rebar fracture and
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613 6 concrete spalling. Therefore, the validated FE models were utilized to further study the effects of
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615 7 boundary conditions, locations of column missing, and story numbers on the varying load transfer
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617 8 mechanism in floors.
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621 9 **4. Detailed Discussion of the Numerical Results**

622 10 *4.1 Load transfer mechanisms of planar multi-story RC frames*

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625 11 As mentioned above, most of the existing tests on progressive collapse research are single-story
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627 12 beam-column substructures or sub-assemblages due to cost and time consideration. However,
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629 13 progressive collapse is a global behavior for a multi-story building, and the load transfer mechanisms
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631 14 may not be the same in each story, especially for the asymmetric structure. Therefore, it is necessary to
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633 15 investigate the various load transfer mechanisms of each floor of the frame model. However, the
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635 16 horizontal constraints of the test specimens were simplified due to the limitation of the cast and testing
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637 17 space. Therefore, to get a more realistic response of structures, a five-span planar frame model with
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639 18 penultimate column loss named BFS-P was built based on the verified modeling techniques in BFS, as
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641 19 shown in Fig. 12. Comparing to BFS, BFS-P has a close-to-reality boundary condition provided by the
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643 20 beams in surrounding bay. Therefore, BFS-P could be the key reference model in this numerical
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645 21 simulation program. To understand the effect of story numbers on the load transfer mechanism of
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647 22 planar RC beam-column substructures subjected to progressive collapse, BFS-P-5F with five stories,
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1 BFS-P-7F with seven stories, and BFS-P-9F with nine stories were also modeled based on the model
2 BFS-P, as shown in Fig. 13.

3 **4.2 Structural resistance of each story**

4 The structural resistance of each story equals the summation of vertical loads on both sides of the
5 beam section located above the removed column. Fig. 14 illustrates the load resistance of each story in
6 specimen BFS-P. As shown in the figure, the load resistance of each story is different after elastic
7 stage. It can be seen that the first story contributed the greatest load resistance when the RCD is less
8 than 133 mm or larger than 220 mm. In terms of CAA capacity, the resistance from the third story is
9 larger than that of the second story and the biggest one is measured in the first story. Regarding the
10 TCA stage, the biggest TCA capacity is also measured in the first story. In general, the assumption of
11 each story demonstrating the same load transfer mechanisms and resistance, which is the basic
12 assumption to use the behavior of a single-story substructure to represent a real multi-story frame, is
13 not accurate for planar frames subjected to a penultimate column missing scenario.

14 Figs. 15, 16, and 17 show the story resistance results of BFS-P-5F, BFS-P-7F, and BFS-P-9F,
15 respectively. Similar to BFS-P, the story resistance began diverging after the elastic stage, and the
16 resistance of the first story is larger than the ones of other stories when the RCD is less than 133 mm
17 or greater than 216 mm. Besides, prior to the fracture of longitudinal reinforcements, the load
18 resistance of the middle stories is quite similar, indicating that the middle stories have similar load
19 transfer mechanisms. To reveal the behavior of multi-story frame subjected to the loss of a column
20 scenario, commonly utilized single-story sub-assembly tests may be insufficient. To the contrary,
21 three sub-assembly tests (top story, one of middle story, and ground story) with proper boundary
22 conditions were required.

4.3 Development of axial forces in beams

To reveal the difference of load transfer mechanism of the beams in each story, the results of the beam axial force of BFS-P were also extracted and presented in Fig. 18. As the axial force throughout each beam is identical, the development of the axial force of the whole beam can be represented by that of one arbitrary section at the beam. Due to asymmetry, the axial force of the beams at different sides of the removed column may be different. Therefore, the axial forces of the beam sections, which are at a distance of 200 mm away from the beam-column interface, were extracted. The labels of L1 to L3 represent the sections at the left side of the removed column (called interior bay) while the labels of R1 to R3 represent the sections at the right side of the removed column (called exterior bay).

In elastic stage, the beams in the first and second stories are in tension while the beam in third story is in compression. These beams worked like a large composite beam under flexure. After elastic stage, as shown in Fig. 18a, the interior-bay beam in the first story (IB-beam-1st) begins to develop compressive force initially and achieved the maximum compressive force of -17.0 kN at a RCD of 108 mm. After that, the axial force of the IB-beam-1st starts to decrease, and changes into tension at a RCD of 208 mm. Different from the IB-beam-1st, the axial force of the interior-bay beam in the second story (IB-beam-2nd) is in tension initially, and it transfers to compression at a RCD of 74 mm. The maximum compressive force of the IB-beam-2nd is -4.0 kN, which is only 23.5 % of the one of the IB-beam-1st. Moreover, the axial force of the IB-beam-2nd transfers to tension again at a RCD of 308 mm. For the interior-bay beam in the third story (IB-beam-3rd), the beam is in compression until the RCD reaches 266 mm, and the maximum compressive force is -10.0 kN, which is 58.8 % of the one of the IB-beam-1st. Besides, the maximum tensile forces of IB-beam-1st, IB-beam-2st, and IB-beam-3st are 27.0 kN, 11.9 kN, and 8.4 kN, respectively. For the exterior bay, as shown in Fig. 18b, the

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783 1 development of the axial force of the exterior-bay beam in the first story (EB-beam-1st) is quite similar
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785 2 to the one of the IB-beam-1st, which is in compression first and finally in tension. Due to interaction of
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787 3 the beam-column elements among stories, the exterior-bay beam in the second story (EB-beam-2st) is
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789 4 in tension first and in compression slight after RCD of 250 mm. However, the exterior-bay beam in
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791 5 third story (EB-beam-3rd) is always in compression during the whole loading history. In a word, the
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793 6 distributions of axial forces in both interior-bay and exterior-bay beams indicate the CAA could
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795 7 develop in the first and third stories, whereas flexural action is the main mechanism of second story to
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797 8 redistribute the gravity load. Moreover, the significant axial tensile forces of the IB-beam-1st and EB-
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799 9 beam-1st in the large deformation stage also illustrate the TCA could develop in the first story beams
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801 10 effectively.
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807 11 Figs. 19, 20, and 21 show the development of the axial forces in the beams of different stories for
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809 12 BFS-P-5F, BFS-P-7F, and BFS-P-9F, respectively. It is observed that the beam axial forces in the
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811 13 middle stories is quite similar. Most of the beam axial forces in the middle stories are mainly in tension
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813 14 first, and the compressive force appears at large deflection stage, indicating that flexural action is the
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815 15 main mechanism of these stories to balance the gravity load. On the other hand, the beam axial forces
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817 16 of the top and bottom stories are similar to the ones of BFS-P. Similarly, the greatest compressive and
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819 17 tensile forces are measured in the first story, which indicates CAA and TCA can develop in the first
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821 18 story effectively. Further parametric study in Section 5.2 will evaluate the accuracy of the conclusion
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823 19 for the frames subjected to interior or corner removal scenarios.
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1 **5. Parametric Study on Planar Multi-Story RC Frame**

2 **5.1 Effect of boundary conditions**

3 As shown in Fig. 2, in the referenced tests [19], the horizontal constraints of the beams in
4 surrounding bay were simplified due to the limitation of the cast and testing space. However, the
5 reliability of the simplification has not been evaluated properly. To quantify the effect of the horizontal
6 restraint stiffness provided by the surrounding bay, four different horizontal restraint stiffness,
7 including 0, 15 kN/mm, 150 kN/mm and rigid, were used for the models of BFS and BFL.

8 Fig. 22 shows the load-displacement curves of BFS and BFL with different boundary conditions.
9 It should be noted that the results of tests are similar to that with rigid restraints for both Specimens
10 BFS and BFL. As shown in Fig. 22, when the horizontal restraint stiffness decreases from rigid to 0,
11 the first peak load (FPL) of BFS and BFL decreases to 87 % and 90 %, respectively, due to weakened
12 CAA. However, reducing the horizontal restraint stiffness is not sensitive to the structural resistance at
13 large deflection stage. Even though there are no spring restraints applied, both BFS and BFL can
14 develop TCA in the initial stage. This is because the remaining two side columns can provide
15 sufficient lateral stiffness to develop TCA initially. However, the TCA weakens due to damage of the
16 side columns later.

17 As shown in Fig. 22a, the FPL of BFS-P, which has a more real boundary condition, is 92 % of
18 that of the BFS. It means that the horizontal restraint stiffness used in the tests may be larger than the
19 real one.

5.2 Effect of location of removed column

For the referenced tests [19], only the scenario of missing a penultimate column is investigated. Two extra numerical models, which were called BFS-I (an interior column was removed in advance) and BFS-C (a corner column was removed in advance), were built to investigate the effects of different column removal scenarios on the load transfer mechanism of each story, as shown in Fig. 23. Fig. 24 shows the decomposition of the load resistance of BFS-I. Similar to BFS-P, the first story achieves the highest initial stiffness and provides the majority of CAA and TCA capacity. However, different from BFS-P, the resistances of the second and third stories are almost the same before RCD reached 285 mm. The difference in the load resistance of these two stories is mainly due to the mobilization of TCA in the second story. As shown in Fig. 25, when RCD exceeds 285 mm, the beams of the second story start to be in tension, indicating the TCA starts to develop in second story too.

For BFS-C, as shown in Fig. 26, the FPL of the first story is also the largest among stories. However, the second story achieves the second largest one, which is different to BFS-P and BFS-I. The different resistance mechanism is due to interaction of the beam-column elements among stories (Vierendeel action).

6. Conclusions

Based on the numerical and parametric studies conducted in this study, the following conclusions are drawn:

1. Comparing with experimental results, it is found that high fidelity numerical models are able to accurately simulate the global behavior of the planar multi-story RC frame subjected to a penultimate column loss scenario.

2. For a planar multi-story RC frame subjected to a penultimate column removal scenario, the

961
962
963 1 load transfer mechanism of each story is not identical. However, when increasing the number of stories,
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965 2 it can be found that the load transfer mechanism of the middle stories is almost the same. Therefore,
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967 3 the behavior of a planar multi-story frame should be equivalently investigated by three types of single-
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970 4 story beam-column assemblies (top-story, middle-story, and ground-story) with proper boundary
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972 5 conditions.

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975 6 3. Horizontal restraint stiffness can significantly affect the development of CAA. Reducing the
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977 7 restraint stiffness of the horizontal springs would decrease the FPL of the frames due to the weakening
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980 8 of CAA. When the horizontal restraint stiffness decreases from rigid to 0, the FPL of BFS and BFL
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982 9 decreases by 87 % and 90 %, respectively. However, horizontal restraint stiffness affecting is
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985 10 insensitive to the development of TCA. Even though spring restraint stiffness reduces to 0, the rest of
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987 11 side columns can provide enough constraints to develop TCA partially.

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990 12 4. It is found from the comparison of the load-displacement curves between the specimens BFS-
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992 13 P and BFS that the load capacity of the Specimen BFS-P is relatively less than that of the Specimen
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995 14 BFS. It means that the horizontal constraints applied on the tests may be stronger than the real one,
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997 15 which will overestimate the capacity of the structure to mitigate progressive collapse.

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999
1000 16 5. Numerical analysis on different column removal scenarios indicates that the beams from a
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1002 17 planar multi-story RC frame subjected to progressive collapse demonstrate different load resistance.
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1004
1005 18 However, the beam in the first story achieves the greatest initial stiffness and load resisting capacity
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1007 19 regardless of the location of removed column.

1008 1009 20 1010 1011 21 **Acknowledgments**

1012
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1023 1 colleges and universities and excellence scholar program in Guangxi (201738). Any opinions, findings
1024
1025 2 and conclusions expressed in this paper do not necessary reflect the view of Natural Science
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1028 3 Foundation of China.

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1187 20 **Captions of tables**

1188
1189 21 **Table 1-** Model parameters of CSCM after adjustment (Units: N, mm and ms)
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1192 23 **Captions of figures**

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1194 24 **Fig. 1**–Reinforcement layout of the Specimen BFS: (a) Elevation view, (b) Cross section of RC frame
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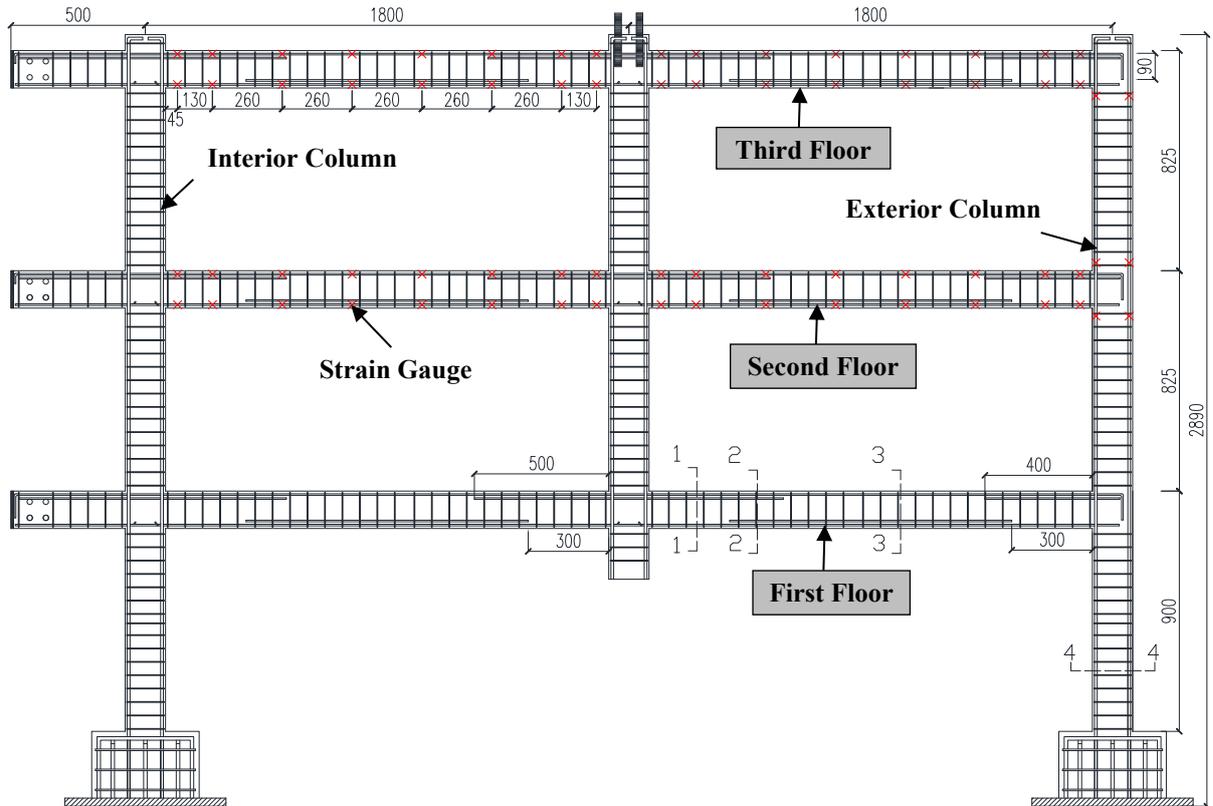
1196 25 Note: Unit in mm, T=Deformed reinforcing bar; R=Plain reinforcing bar
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1203 1 **Fig. 2**–Test setup and instrumentation
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1208 4 **Fig. 5**–Comparisons of different mesh sizes
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1211 6 **Fig. 7**–Unconfined uniaxial stress-strain relationship of concrete for 32.1 MPa based on CSCM
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1213 7 **Fig. 8**–Comparison of the load-displacement between simulation and test: (a) BFS, (b) BFL
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1215 8 **Fig. 9**– Comparison of the horizontal movement response from numerical and test: (a) BFS, (b) BFL
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1217 9 **Fig. 10**–Comparisons of damage patterns of BFS: (a) FEM, (b) Test
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1219 10 **Fig. 11**–Comparisons of damage patterns of BFL: (a) FEM, (b) Test
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1221 11 **Fig. 12**–Numerical model of BFS-P
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1223 12 **Fig. 13**–Numerical models of number of different floors: (a) BFS-P-5F, (b) BFS-P-7F, (c) BFS-P-9F
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1225 13 **Fig. 14**–Load resistance of each story of BFS-P
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1227 14 **Fig. 15**–Load resistance of each story of BFS-P-5F
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1231 16 **Fig. 17**–Load resistance of each story of BFS-P-9F
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1233 17 **Fig. 18**–Development of beam axial forces of BFS-P: (a) Left side of removed column, (b) Right side
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1237 19 **Fig. 19**–Development of beam axial forces of BFS-P-5F: (a) Left side of removed column, (b) Right
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1241 21 **Fig. 20**–Development of beam axial forces of BFS-P-7F: (a) Left side of removed column, (b) Right
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1245 23 **Fig. 21**–Development of beam axial forces of BFS-P-9F: (a) Left side of removed column, (b) Right
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1247 24 side of removed column
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1249 25 **Fig. 22**–Comparison of different boundary conditions: (a) BFS, (b) BFL,
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1251 26 **Fig. 23**–Numerical models of different column loss: (a) BFS-I, (b) BFS-C
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1253 27 **Fig. 24**–Load resistance of each story of BFS-I
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1257 29 **Fig. 26**–Load resistance of each story of BFS-C
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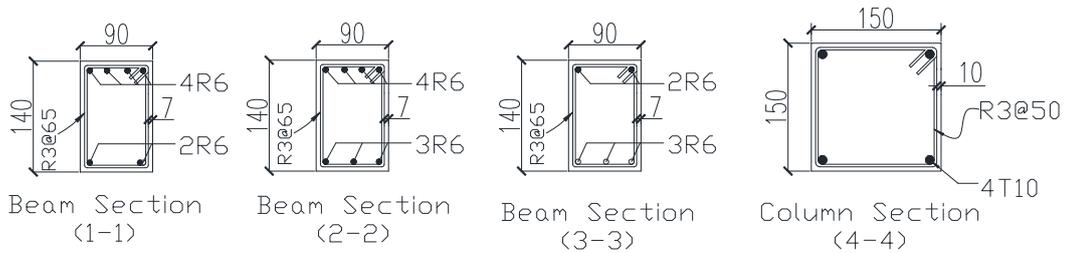
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Table 1
Model Parameters of CSCM after Adjustment (Units: N, mm and ms)

MID	RO	NPLOT	INCRE	IRATE	ERODE	RECOV	ITRETRC
1	0.00232	1	0.0	0	1.10	0.0	0
PRED							
0							
G	K	ALPHA	THETA	LAMDA	BETA	NH	CH
7065	7738	14.788	0.3029	10.5	0.01929	0	0
ALPHA1	THETA1	LAMDA1	BETA1	ALPHA2	THETA2	LAMDA2	BETA2
0.74735	0.001102	0.17	0.06855	0.66	0.001323	0.16	0.06855
R	XD	W	D1	D2			
5.0	91.5	0.05	2.5e-04	3.492e-07			
B	GFC	D	GFT	GFS	PWRC	PWRT	PMOD
100.0	4.575	0.1	0.04575	0.02288	5.0	1.0	0.0
ETA0C	NC	ETA0T	NT	OVERC	OVERT	SRATE	REPOW
0	0	0	0	0	0	0	0



(a)



(b)

Fig. 1–Reinforcement layout of the Specimen BFS: (a) Elevation view, (b) Cross section of RC frame

Note: Unit in mm, T=Deformed reinforcing bar; R=Plain reinforcing bar

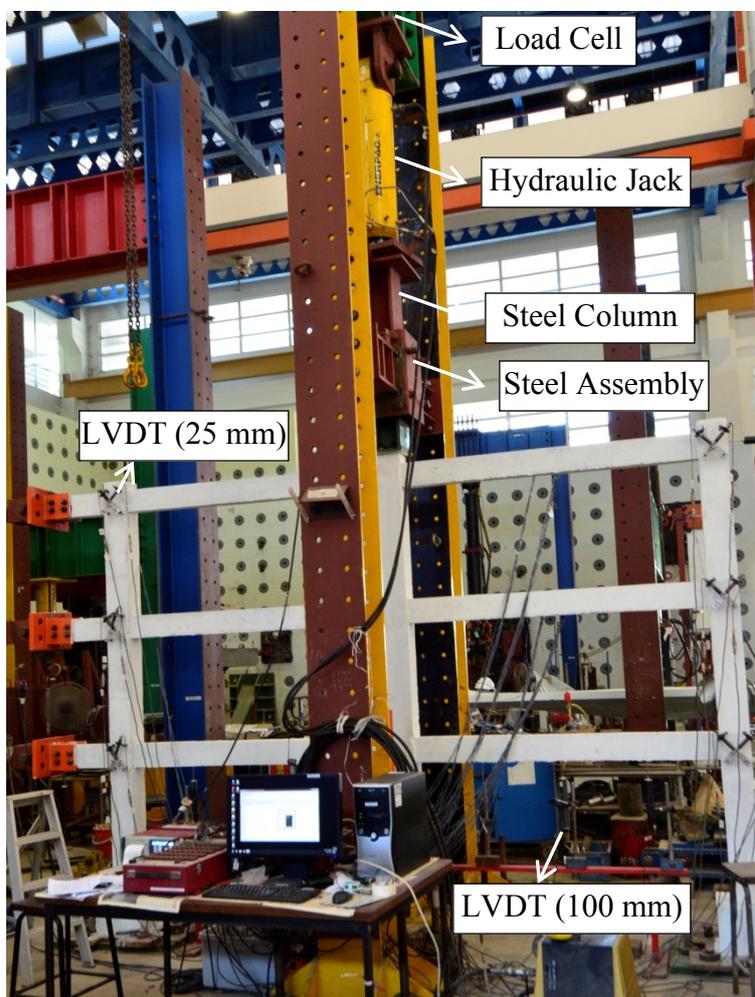


Fig. 2–Test setup and instrumentation

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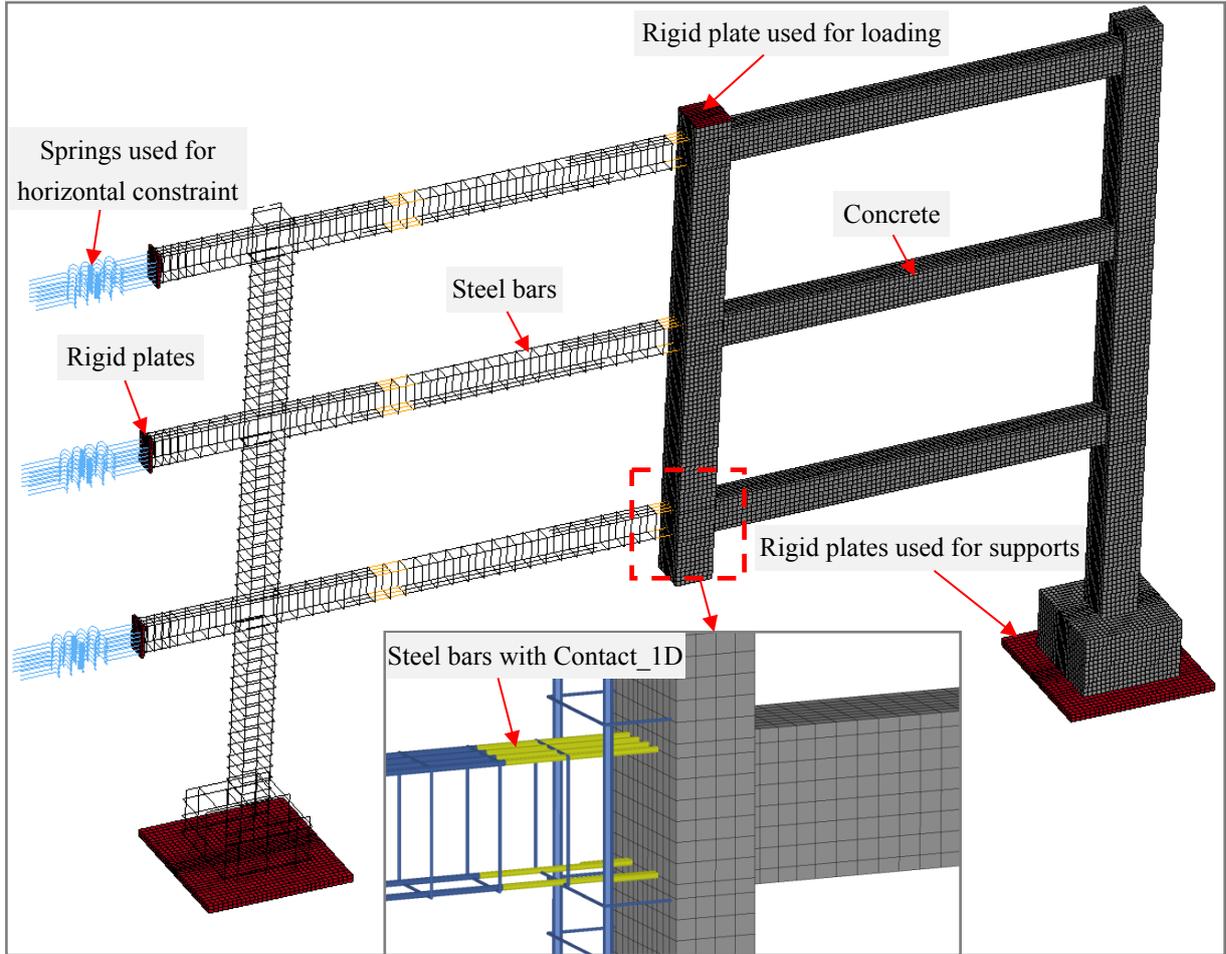


Fig. 3–Geometric model of Specimen BFS

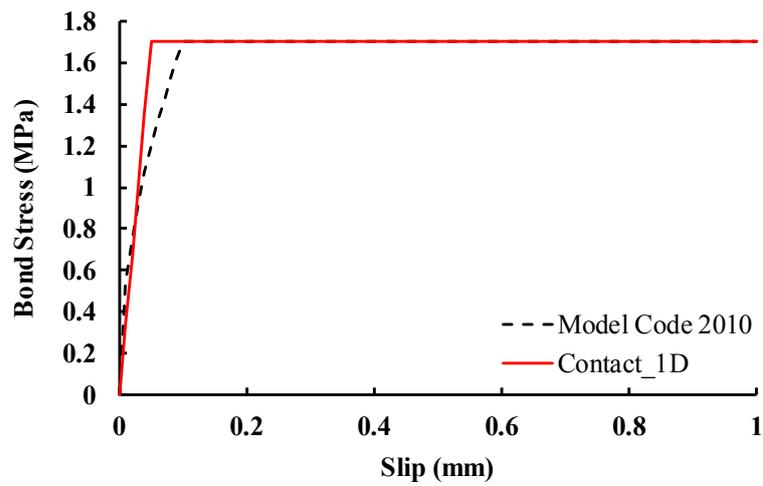


Fig. 4–Comparison of bond-slip relationship between Model Code 2010 and Contact_1D

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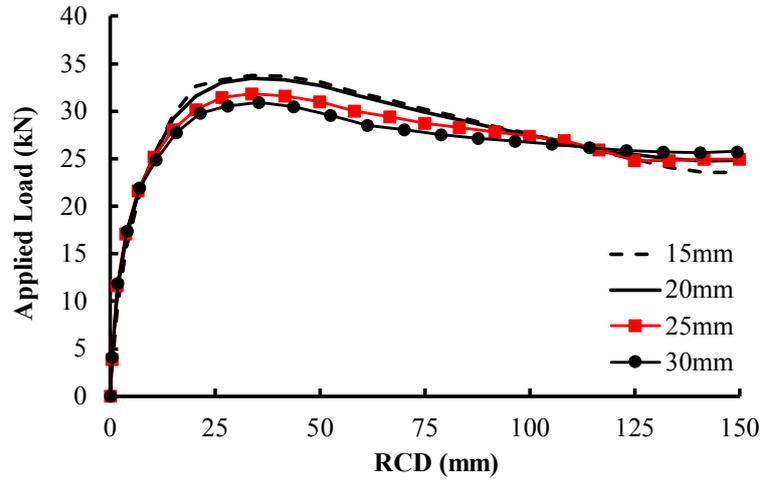


Fig. 5–Effects of different mesh sizes

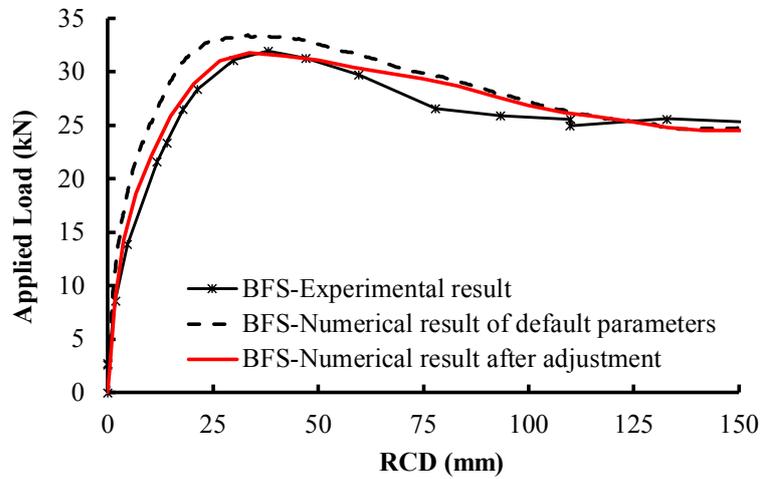


Fig. 6–Effects of different concrete input parameters

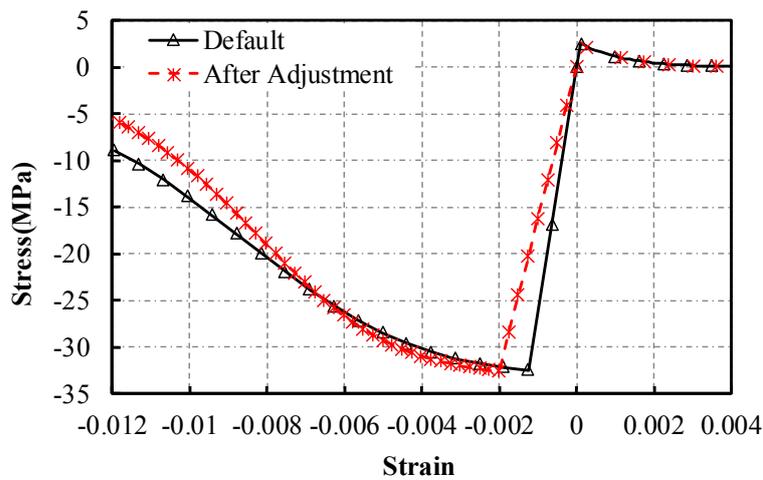
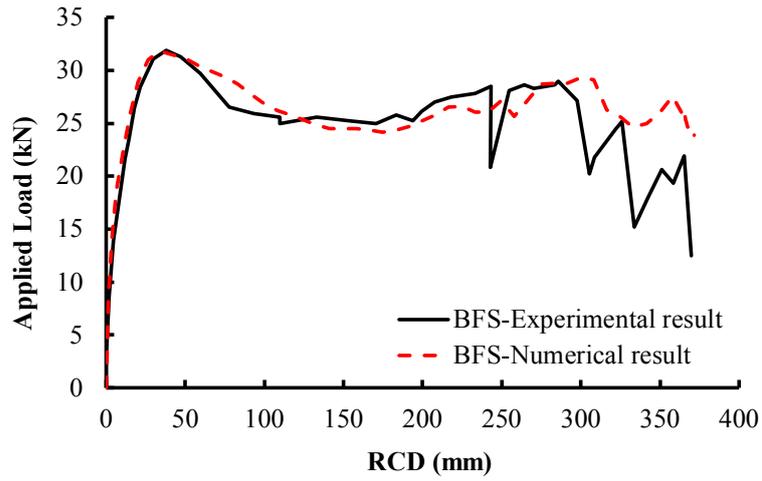
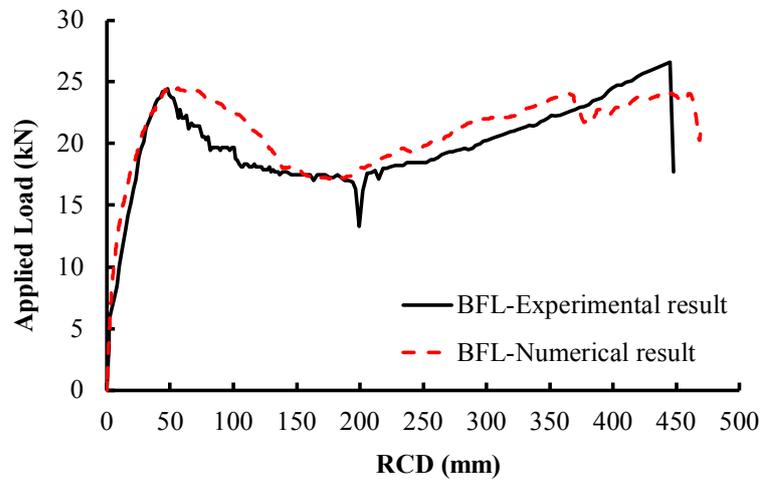


Fig. 7–Unconfined uniaxial stress-strain relationship of concrete for 32.1 MPa based on CSCM

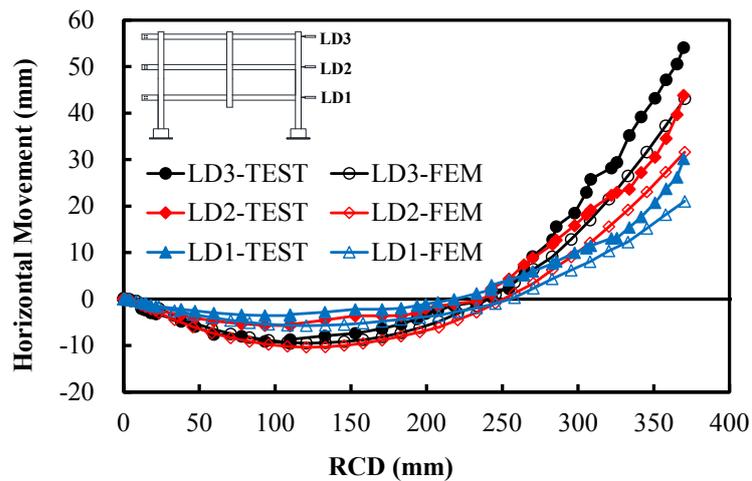


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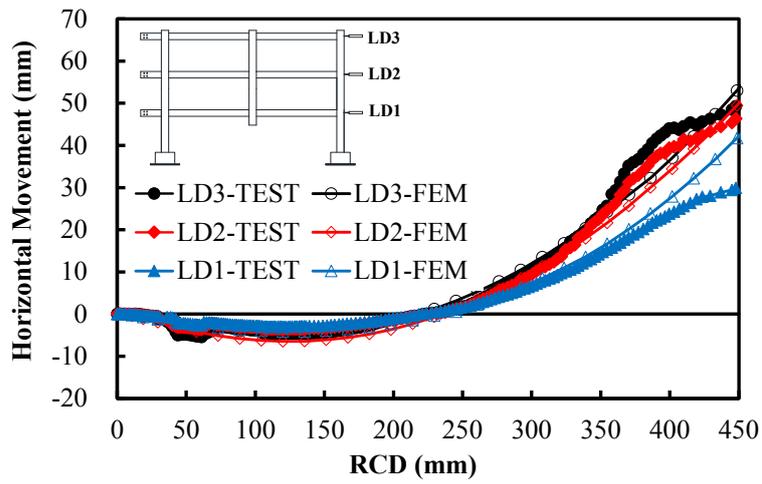


(b)

Fig. 8–Comparison of the load-displacement response from numerical and test: (a) BFS, (b) BFL

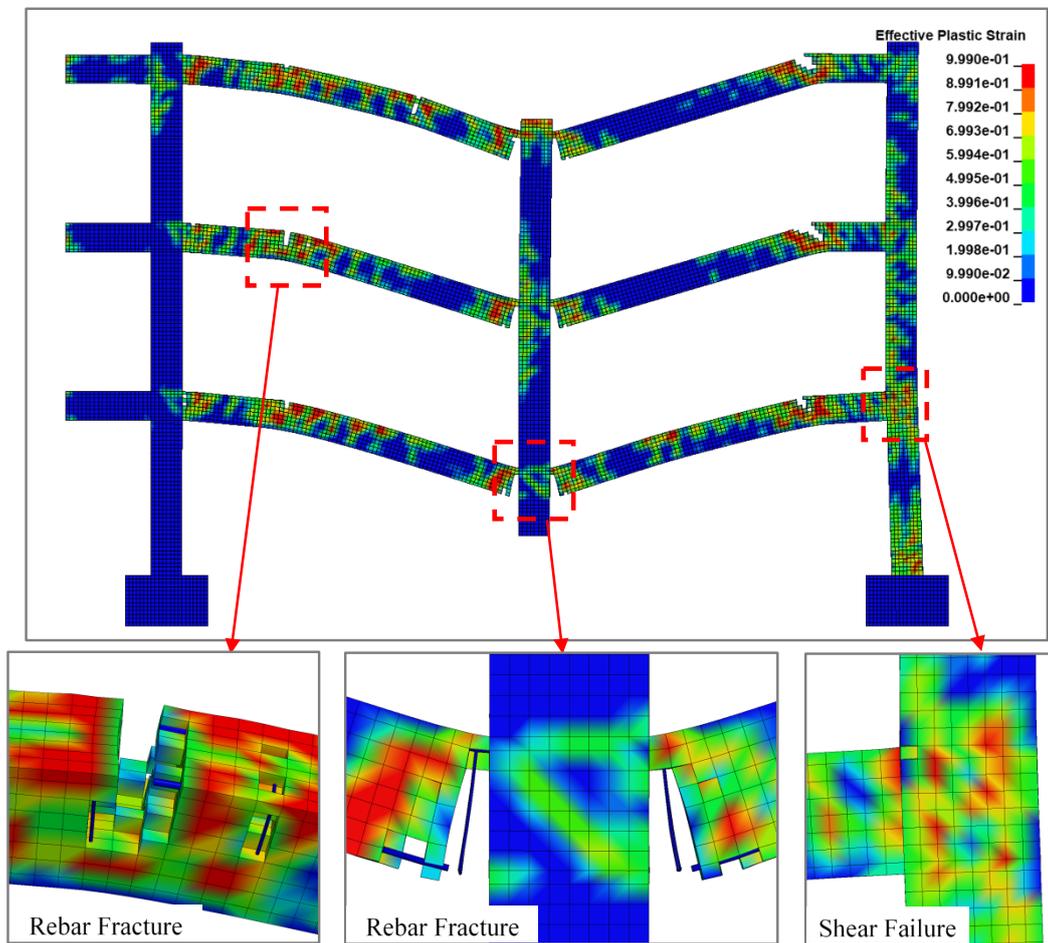


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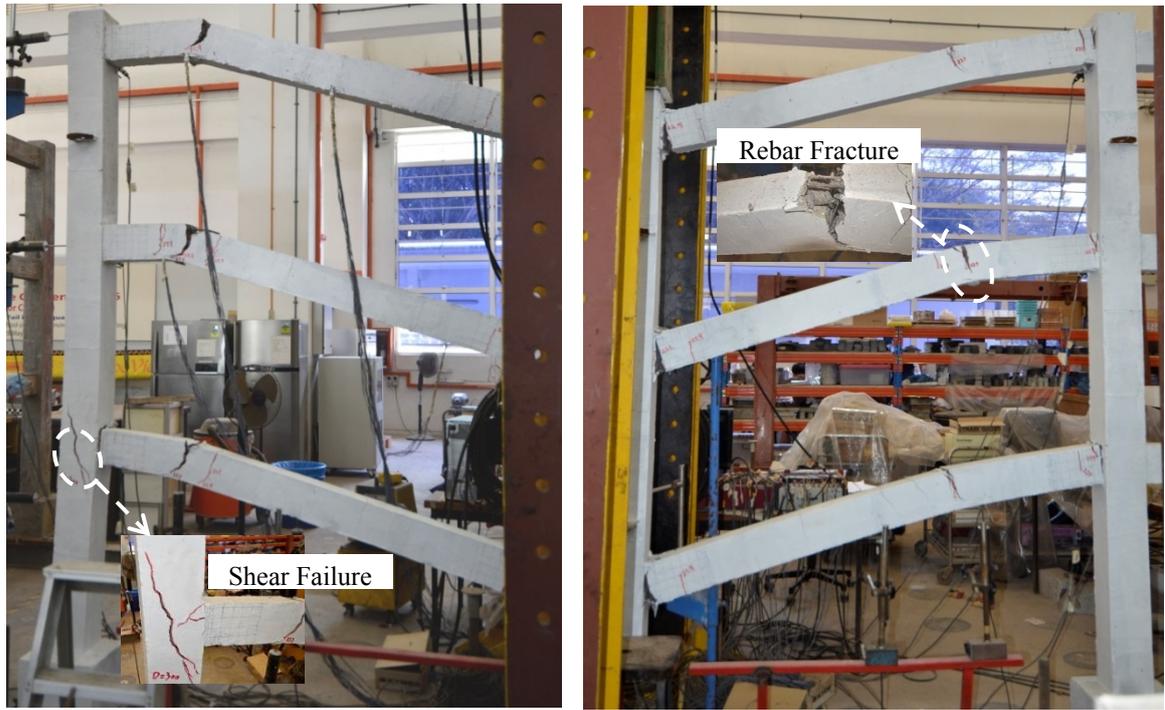


(b)

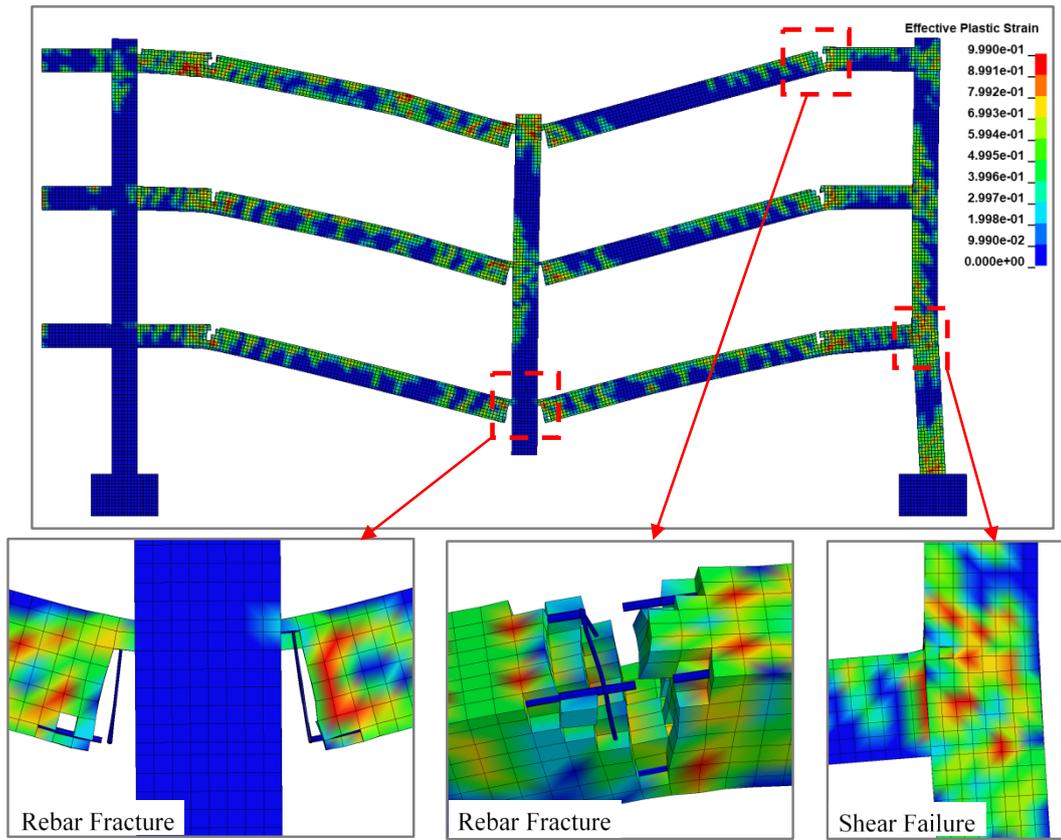
Fig. 9–Comparison of the horizontal movement response from numerical and test: (a) BFS, (b) BFL



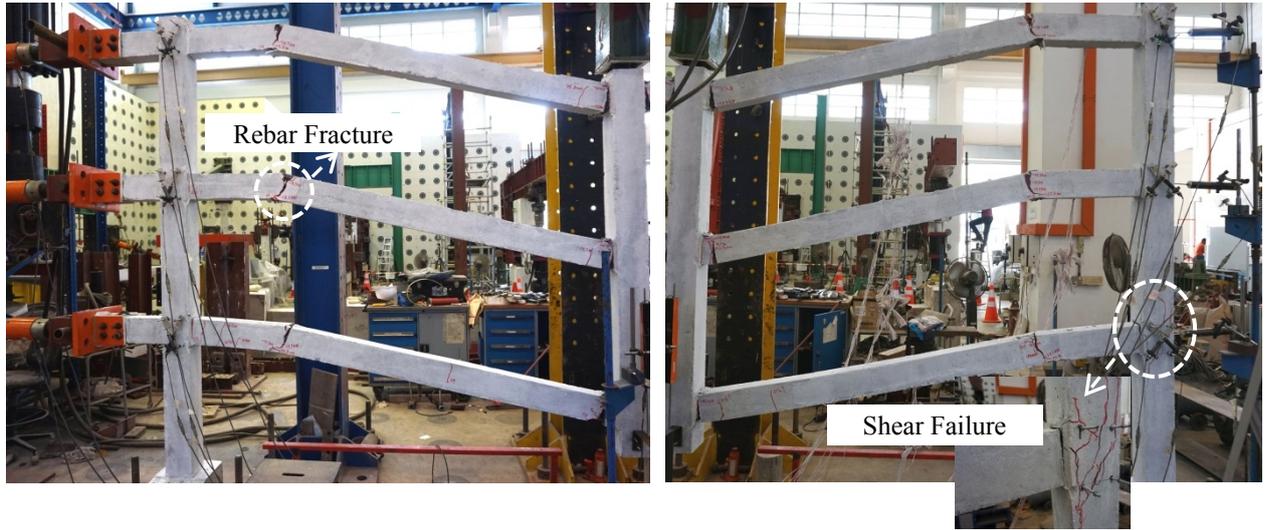
(a)



(b)
Fig. 10–Comparisons of failure mode of BFS: (a) FEM, (b) Test



(a)



(b)

Fig. 11–Comparisons of failure mode of BFL: (a) FEM, (b) Test

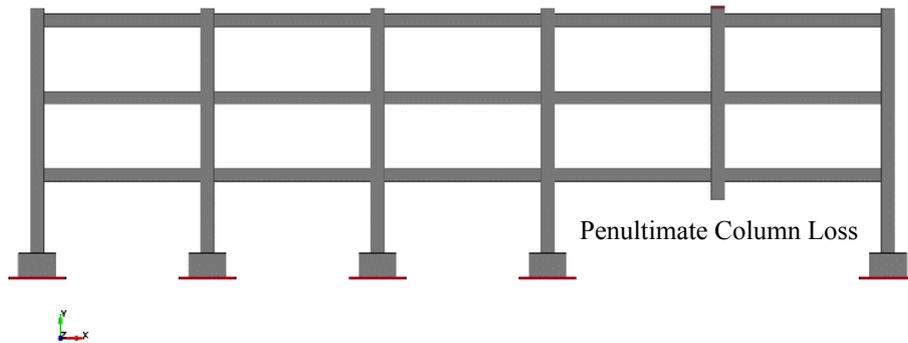
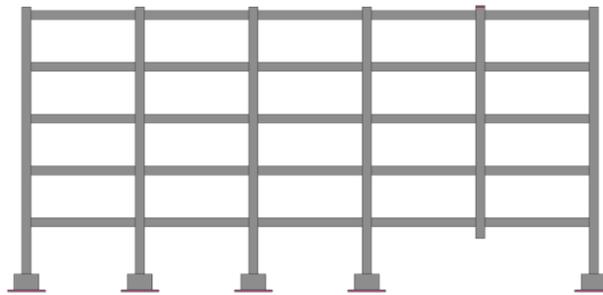
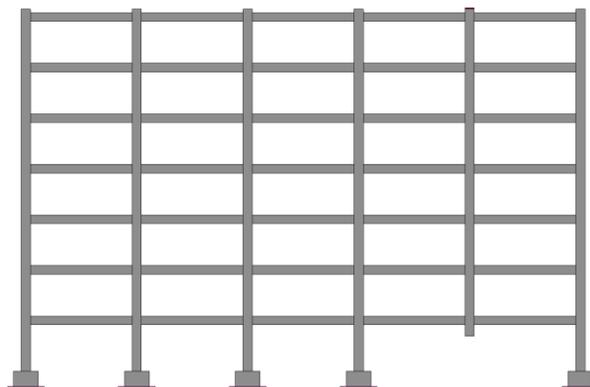


Fig. 12–Numerical model of BFS-P



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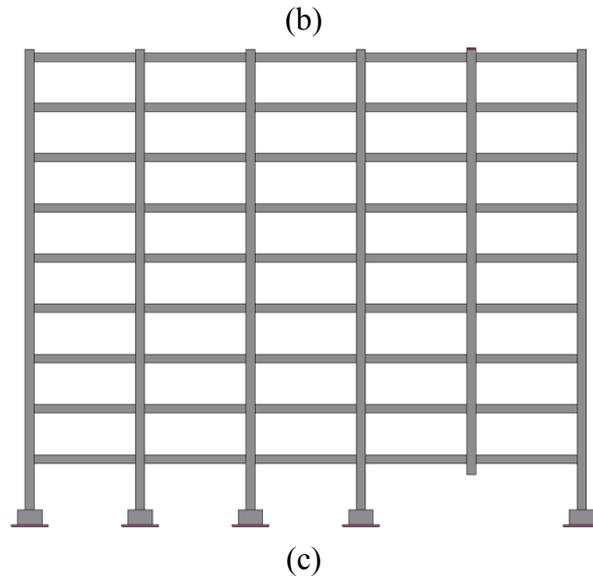


Fig. 13–Numerical models of number of different floors: (a) BFS-P-5F, (b) BFS-P-7F, (c) BFS-P-9F

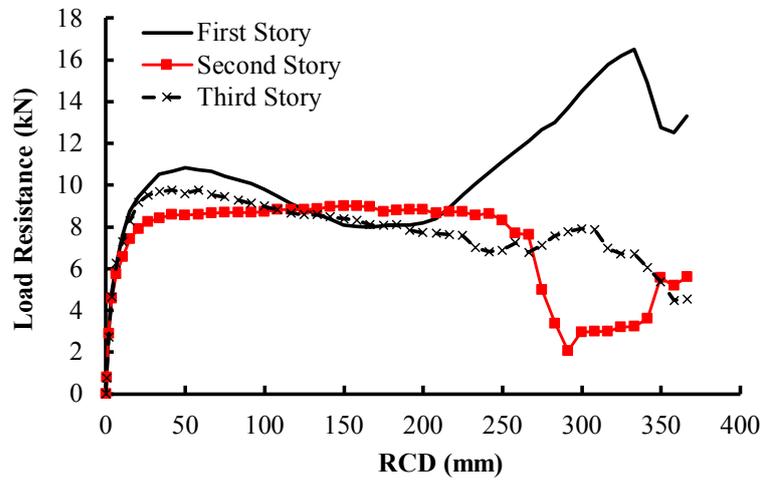


Fig. 14–Load resistance of each story of BFS-P

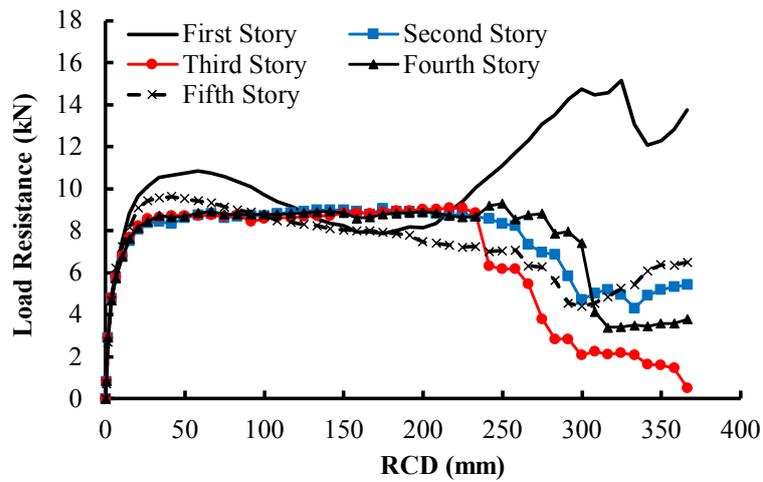


Fig. 15–Load resistance of each story of BFS-P-5F

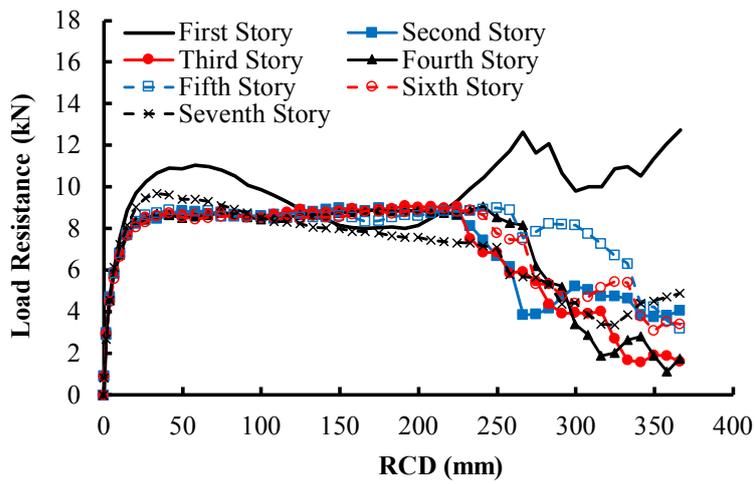


Fig. 16–Load resistance of each story of BFS-P-7F

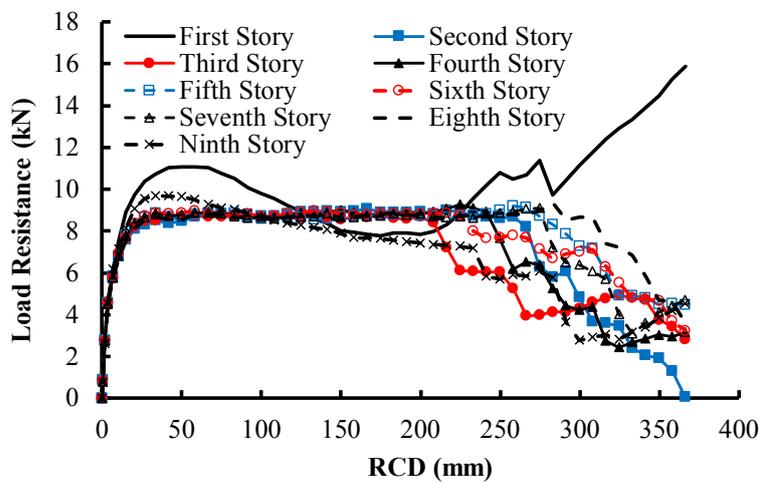
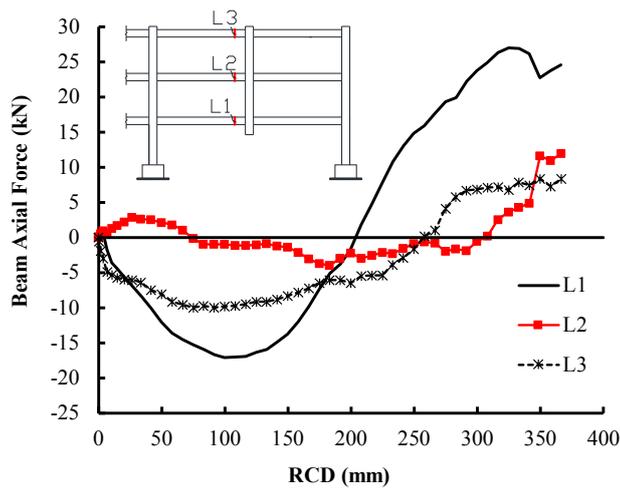
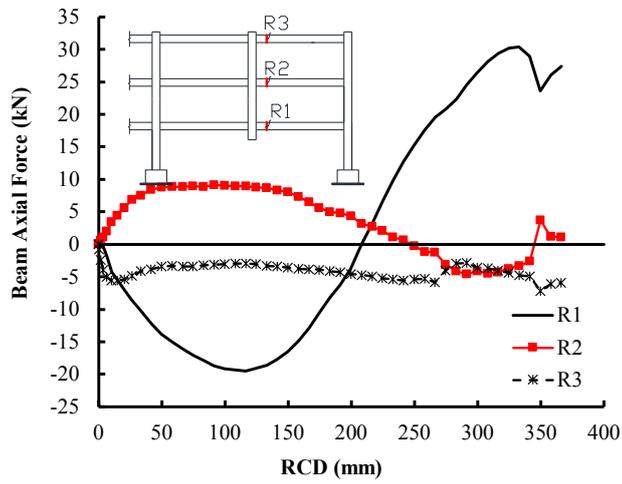


Fig. 17–Load resistance of each story of BFS-P-9F

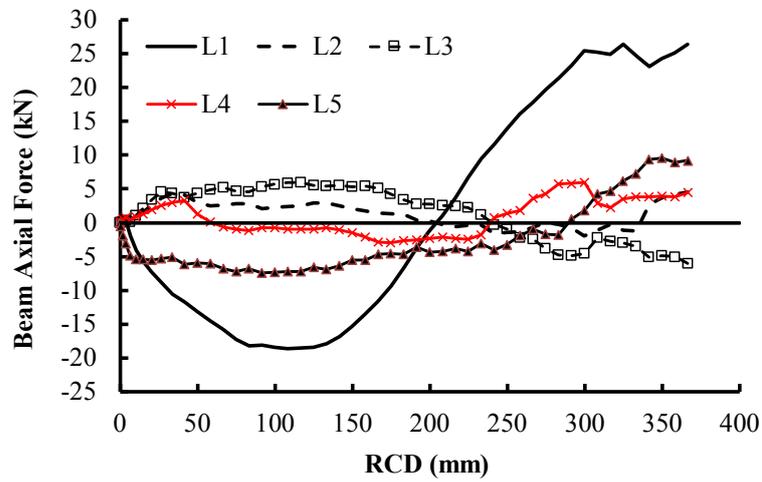


(a)

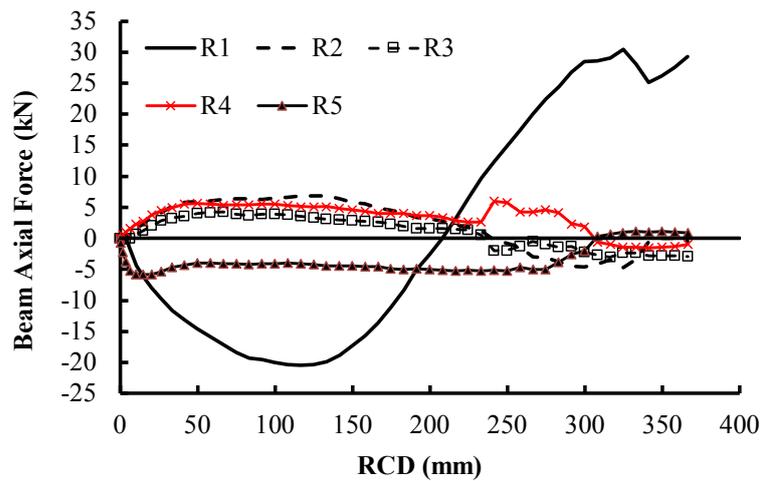


(b)

Fig. 18–Varying of beam axial forces of BFS-P: (a) left side of removed column, (b) right side of removed column

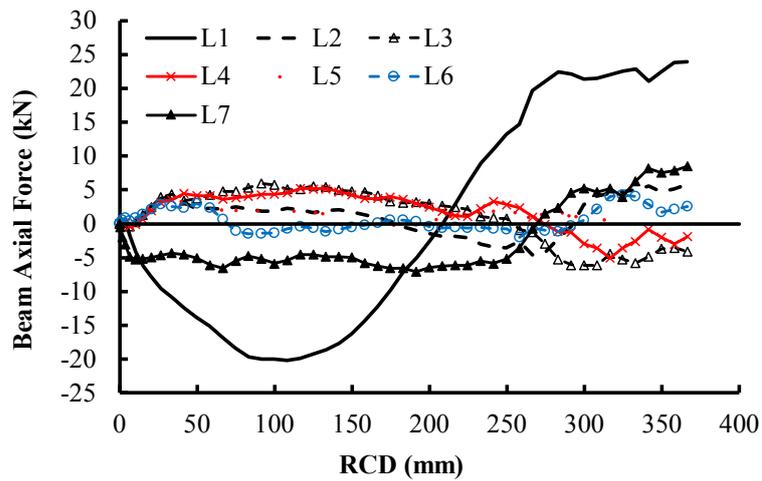


(a)

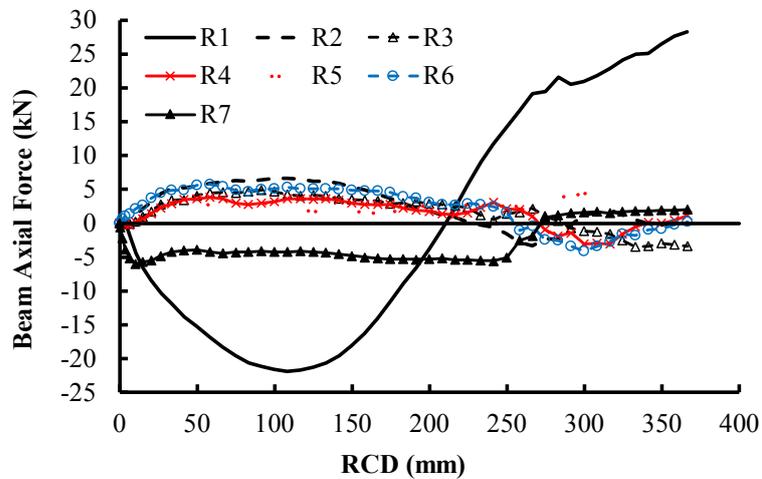


(b)

Fig. 19–Varying of beam axial forces of BFS-P-5F: (a) left side of removed column, (b) right side of removed column

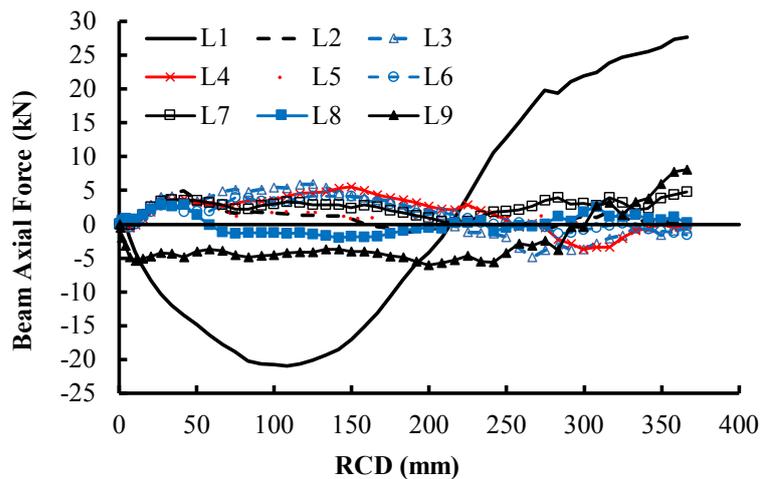


(a)

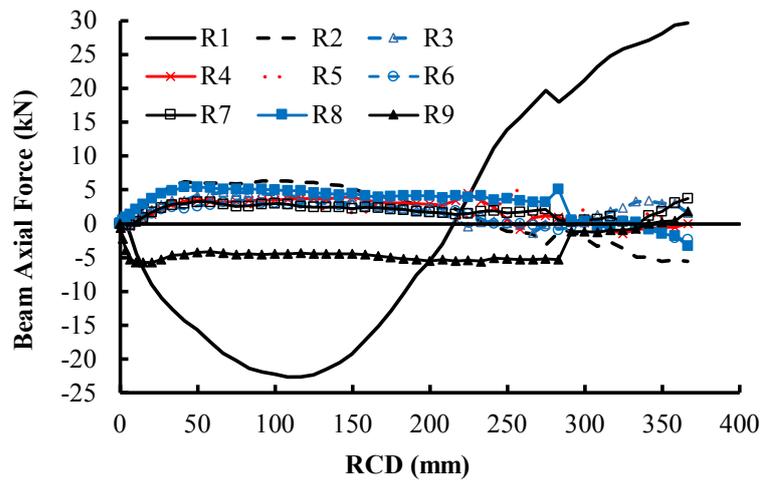


(b)

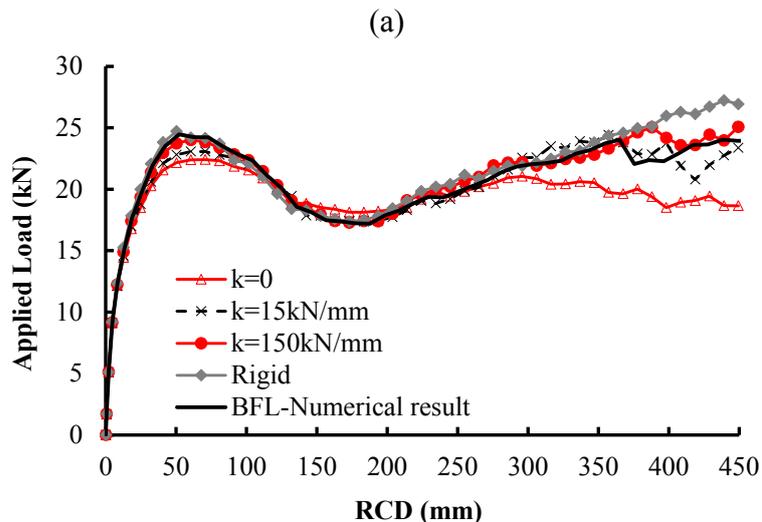
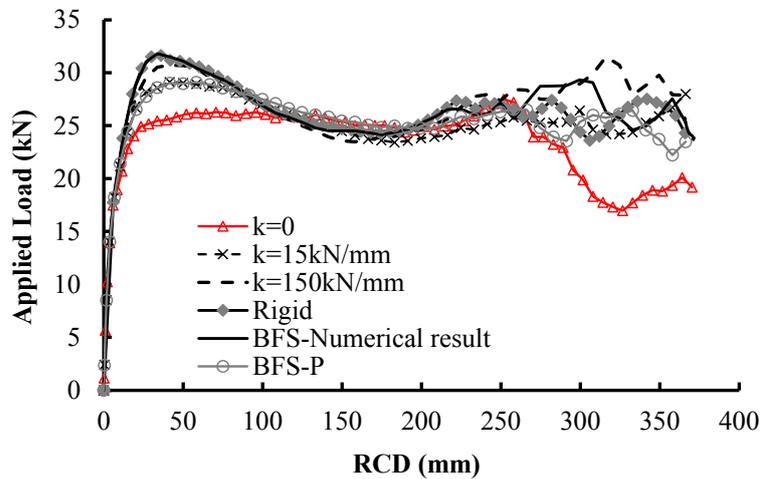
Fig. 20—Varying of beam axial forces of BFS-P-7F: (a) left side of removed column, (b) right side of removed column



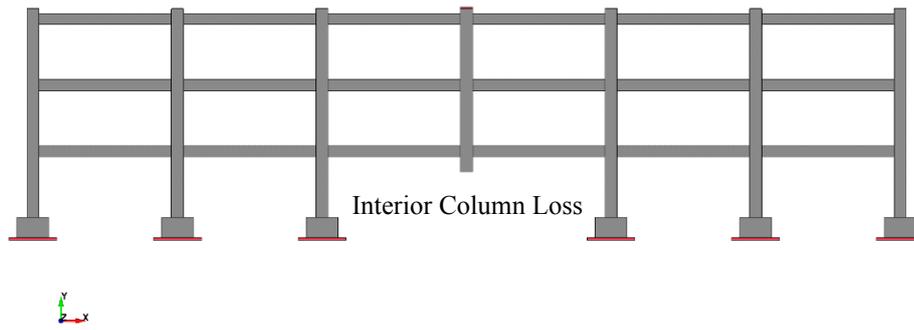
(a)



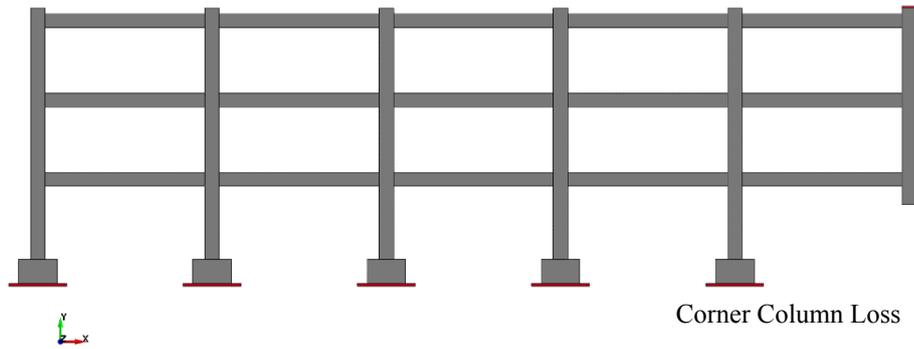
(b)
Fig. 21—Varying of beam axial forces of BFS-P-9F: (a) left side of removed column, (b) right side of removed column



(a)
Fig. 22—Effects of horizontal stiffness of the constraints: (a) BFS, (b) BFL



(a)



(b)

Fig. 23–Numerical models of different column loss: (a) BFS-I, (b) BFS-C

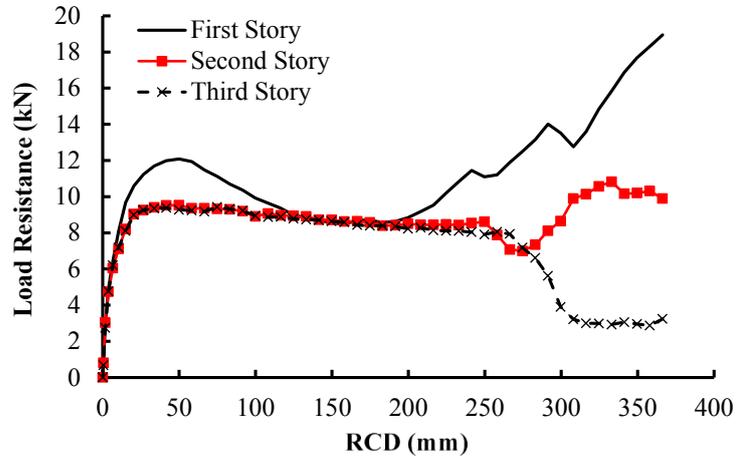


Fig. 24–Load resistance of each story of BFS-I

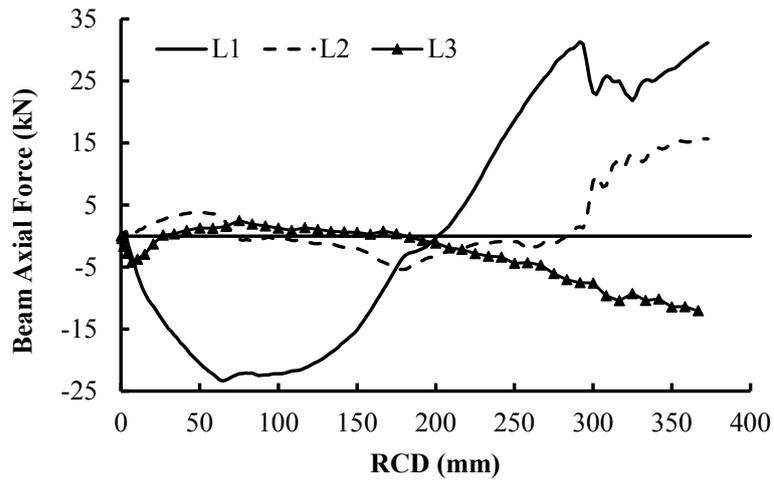


Fig. 25–Varying of axial force of the beams in different story of BFS-I

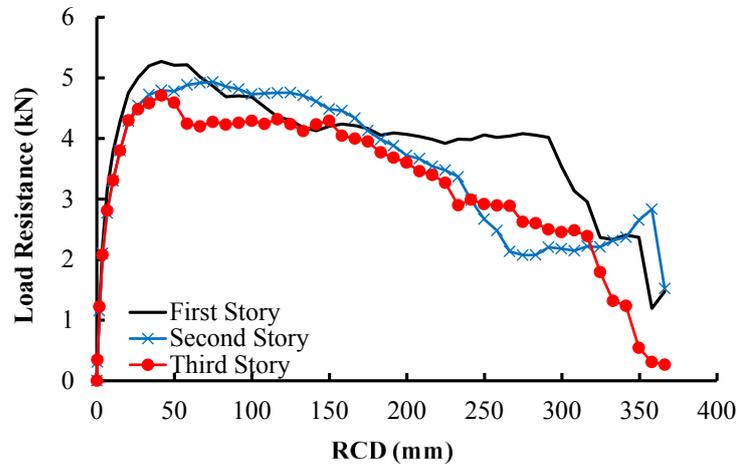


Fig. 26–Load resistance of each story of BFS-C