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# Numerical Investigation on Load Redistribution Capacity of Flat Slab Substructures to Resist Progressive Collapse

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**Abstract:** To study the load redistribution capacity of reinforced concrete (RC) flat slab structures subjected to a middle column loss scenario, high fidelity finite element (FE) models were built using commercial software LS-DYNA. The numerical models were validated by experimental results. It is found that the continuous surface cap model (CSCM) with an erosion criterion considering both the maximum principal and shear strain could effectively predict the punching shear failure at slab-column connections. The validated FE models were employed to investigate the effect of boundary conditions, amount of integrity reinforcement, and slab thickness on the load redistribution capacity of flat slab structures. Furthermore, multi-story RC flat slab substructures were built to capture the load redistribution behavior of different floors. Parametric studies indicate that ignoring the constraints from surrounding slabs may underestimate the load redistribution capacity of the flat slab substructures. Therefore, it is suggested that in future numerical or experimental studies, rigid horizontal constraints should be applied at the slab edge of the substructure to well represent the constraints from surrounding slabs. In addition, it is also found that the amount of integrity reinforcement would significantly affect the post-punching performance of flat slab structures. It is suggested that the minimum integrity reinforcement ratio should be 0.63 %.

**Keywords:** Progressive collapse; Flat slab substructures; Quasi-static; Punching shear; Load resisting mechanism

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## 26    **1. Introduction**

27            An initial local failure of supporting components due to accidents may lead to a disproportionate  
28 collapse of the structure, which is defined as disproportionate collapse or progressive collapse. The  
29 consequence of casualties caused by progressive collapse event is very tragic. Progressive collapse  
30 attracted public attention first after the collapse of Ronan Point apartment in 1968 and raised  
31 concerns by design regulators after the catastrophic consequences of the Murrah Federal Building in  
32 Oklahoma City in 1995 and World Trade Centre in New York City in 2001. Several design  
33 guidelines (DoD 2009 [1]; GSA 2003 [2]; ASCE/SEI-10 2010 [3]; COST Action TU0601 2008 [4])  
34 were proposed accordingly. Moreover, these progressive collapse incidents also attracted a large  
35 number of researchers to investigate the load redistribution behavior and load resisting mechanisms  
36 of the structures subjected to different column removal scenarios. In the past decade, several studies  
37 have been conducted to study progressive collapse based on the alternative load path method  
38 stipulated in DoD [1] and GSA [2].

39            Su et al. [5] tested twelve reduced-scale RC beam-column sub-assemblages to investigate the  
40 effects of beam reinforcement ratio, span-to-depth ratio, and loading rate on the compressive arch  
41 action of RC frames. Two one-half scaled sub-assemblages with seismic and non-seismic detailing  
42 were tested by Yu and Tan [6] to evaluate the effects of seismic design on behavior of RC frames in  
43 resisting progressive collapse. Feng et al. [7] evaluated the behavior of precast concrete structures to  
44 resist progressive collapse by using a three-dimensional FE model. Fascetti et al. [8] proposed a  
45 procedure to evaluate the robustness of RC frame against progressive collapse based on a macro-  
46 model simulation. Livingston et al. [9] carried out a series of pushdown analysis to quantify the  
47 effects of structural characteristics (e.g. axial stiffness at the beam boundaries, amount of integrity  
48 reinforcement at bar cut-off locations, etc) on progressive collapse resistance of frames. High fidelity  
49 solid-element-based numerical models were used by Yu et al. [10] to investigate the robustness of  
50 RC beam-slab substructures under perimeter column removal scenarios. Shan et al. [11] tested two  
51 one-third scale, four-bay by two-story RC frame to investigate the effects of infilled wall on the load  
52 resisting mechanisms of RC frames. Based on their tests, it was concluded that infilled walls could

enhance the load resisting capacity of RC frames significantly due to more alternative load paths provided. However, the infilled walls might decrease the ductility of the frames. Sadek et al. [12] tested four (two RC and two steel) full-scale beam-column sub-assemblages subjected to the loss of an interior column to provide insight into the mobilization of catenary action. Qian and Li [13] tested two series of six specimens (with and/or without RC slab) to quantify the contribution of RC slabs in resisting progressive collapse under a corner column loss scenario. It was found that RC slabs could improve the ultimate load resisting capacity by 63 %. Pham et al. [14] investigated the effect of different loading patterns and boundary conditions on tensile membrane action (TMA) of beam-slab systems. Lu et al. [15] and Ren et al. [16] conducted a number of one-third scaled specimens to investigate the effects of RC slabs on the behavior of RC frames to resist progressive collapse caused by either an edge or an interior column loss scenario. Feng et al. [17] used the probability density evolution method to evaluate the robustness of RC beam-column sub-assemblage under a column missing scenario.

Flat slab floor system has been widely used in tall and multi-story buildings, due to its long span and small thickness features. However, there is relatively little attention paid on flat slab structures to resist progressive collapse. For flat slab structures, a column loss leads to load redistribution, result in the increase of bending moment and shear force at adjacent slab-column connections significantly. The likelihood of progressive collapse increases as no load redistribution in beams can be triggered in flat slab structures. This may cause collapse of the buildings such as of the incidents of Sampoong Department Store at Seoul, South Korea. Russell et al. [18] tested seven 1/3 scaled RC flat slab substructures subjected to the quasi-static or dynamic loading regimes. The experimental results showed that flat slabs could redistribute the loads effectively after a column lost. However, punching shear failure was a critical issue must be addressed seriously. Qian and Li [19, 20] and Ma et al. [21] conducted several series of tests to investigate the load resisting mechanisms and quasi-static behaviour of RC flat slab structures subjected to the loss of corner or interior column scenarios. Keyvani et al. [22] developed a new finite element modeling technique to simulate punching and post-punching behavior of flat plates. Moreover, the effects of compressive membrane action (CMA)

80 on load resisting capacity of flat plate structures were also investigated by Keyvani et al. [22]. Liu et  
81 al. [23] proposed a macromodel for slab-column connections, which could be used to simulate the  
82 behaviour of flat slab or flat plate structures in resisting progressive collapse. Peng et al. [24] carried  
83 out a series of dynamic tests to study the dynamic response of flat plate substructure subjected to  
84 instantaneously removal of an exterior column. Qian et al. [25] conducted experimental and  
85 numerical studies to evaluate the dynamic response of flat slab structures subjected to different  
86 extents of initial local damage (one-column or two-column removal).

87 Due to the complexity of testing on multi-panel flat slab structures, the majority of existing  
88 experimental tests on progressive collapse resistance are single-storey substructures or sub-  
89 assemblages with simplified boundary condition (applying weights at the overhang to simulate the  
90 constraints from surrounding components [19-21, 24] or ignoring the constraints from surrounding  
91 components [18]), which is different from the real conditions in a building. It is necessary to conduct  
92 further studies to evaluate the effects of boundary condition on the behavior of flat slab substructures  
93 to mitigate progressive collapse. Moreover, due to excessive time and high cost for experimental  
94 studies, some of critical parameters could not be investigated by experimental studies. Therefore,  
95 developing an accurate numerical simulation method is imperative. In this paper, numerical  
96 simulation based on high fidelity FE models are developed using LS-DYNA. The FE models are  
97 validated by test results. Then, the validated FE models are used to quantify the effect of surrounding  
98 slabs and upper floors on the load redistribution capacity of flat slab substructure. In addition, for  
99 multi-storey flat slab buildings, the load redistribution ability of each floor was evaluated  
100 individually to reveal the difference of loading resisting mechanisms and load resisting contribution  
101 of each floor. Finally, the effects of integrity reinforcement and slab thickness on the load  
102 redistribution capacity of flat slab substructures are also investigated.

## 103    **2. Experimental program and numerical validation**

### 104    *2.1. Brief of experimental program*

105        To determine the possible load resisting mechanisms in flat slab structures to resist progressive  
106 collapse, a test program was conducted by Qian and Li [20]. An experimental program included three  
107 1/4 scaled multi-panel flat slab substructures. The prototype of these specimens was designed  
108 according to provisions of ACI 318-11 [26]. The total dead load (DL) including the self-weight of the  
109 slab and the weights of infilled walls was assumed to be 8.0 kPa. The live load (LL) was assumed to  
110 be 3.0 kPa. Two specimens (ND and WD) from Qian and Li [20] are used to validate numerical  
111 models. These two flat slab substructures have identical dimensions and reinforcement details. One  
112 of the specimens has enhanced punching shear capacity due to the drop panels at slab-column  
113 connections. Table 1 gives the specimen properties while Fig. 1 shows the dimensions and  
114 reinforcement details of Specimen WD. As it can be seen from the figure, Specimen WD consists of  
115 a slab with dimension of 3750 mm×3750 mm×55 mm, nine columns including one interior column  
116 and eight surrounding columns, and nine drop panels with size of 450 mm×450 mm×35 mm. The  
117 cross-section of columns was 200 mm×200 mm. The interior column was reinforced by 4-T13.  
118 However, the surrounding columns were reinforced by 8-T13 to further enhance their strength and  
119 stiffness. The drop panel was reinforced with a single layer mesh of R6@80 mm. The RC flat slab  
120 was reinforced using two layers of R6@250 mm mesh. Moreover, in the bottom layer, integrity rebar  
121 of 3R6 was installed within the column reinforcing cage in orthogonal directions, to meet the  
122 detailing requirements of ACI 381-11 [26] (more than two reinforcements passing through the  
123 column cage as integrity rebar). T13 and R6 represent the deformed rebar with diameter of 13 mm  
124 and plain rebar with diameter of 6 mm, respectively. The yield strength and ultimate strength of R6  
125 were 500 MPa and 617 MPa, and those of T13 were 529 MPa and 608 MPa. After 28 days curing,  
126 the measured concrete compressive strength of Specimens ND and WD was 22.5 MPa and 22.3 MPa,  
127 respectively.

128 Test setup and instrumentation layout of Specimen WD are shown in Fig. 2. As shown in the  
129 figure, the specimen was supported by eight steel legs. A special load distribution rig was designed to  
130 equivalently replicate the uniform distributed load (UDL). The detailing of the load distributed rig is  
131 illustrated in Fig. 3. It has three rigid beams, four triangle steel plates, and twelve small steel plates.  
132 Between the secondary steel beams and triangle steel plates, hemisphere balls and socket joints were  
133 utilized to ensure the load can be vertically applied during the tests, even at the stage that the large  
134 deformation of slabs. Moreover, a hydraulic jack with a stroke of 600 mm was utilized to apply loads  
135 and a steel assembly (Item 4 in Figure 2) was designed to ensure that the applied load keeps  
136 vertically. More details about the test program please refer to Qian and Li [20].

## 137 2.2. Details of numerical model

138 In this study, high fidelity finite element model was developed to investigate the difference of  
139 load resisting mechanisms on each floor of a multi-storey flat slab structure subjected to a middle  
140 column removal scenario. The explicit software LS-DYNA [27] was employed due to its numerical  
141 stability and sufficient availability of constitutive models. Although quasi-static behavior of the flat  
142 slab substructure was focused on in this study, explicit solver was adopted to avoid divergence  
143 problem at large deformation stage.

### 144 2.2.1 Element type

145 Fig. 4 shows the geometrical model of WD. For another model ND, it is identical to WD in  
146 slabs, columns, and reinforcement details, except no drop panels are modelled. To simulate the  
147 boundary conditions more close to real test conditions, eight steel legs and the load distribution rig  
148 were also simulated in FE modeling. The element of concrete adopted in this study is 8-node solid  
149 elements with reduced integration. This reduced integration element can save computational time on  
150 the premise of accuracy when hourglass control is well defined. To ensure the hourglass energy was  
151 less than 10 % of the total internal energy, Flanagan-Belytschko stiffness form with exact volume  
152 integration was used for solid elements. Thus, the hourglass coefficient was defined as 0.002. A 2-  
153 node Hughes-Liu beam element with  $2 \times 2$  Gauss quadrature integration was employed to simulate the

reinforcements. This Hughes-Liu beam element could effectively simulate the mechanical behavior of reinforcement bars, such as axial force, bi-axial bending, and transverse shear. Moreover, the load distribution rig and steel supports were also modeled by explicit solid element.

For the connection between reinforcement and concrete, previous studies [10, 14] had proved that the assumption of perfect bonding between slab reinforcement and concrete could ensure enough accuracy to simulate the behavior of RC component subjected to a column removal scenario. As a result, perfect bonding between concrete and reinforcement was assumed by keyword \*Constrained\_Lagrange\_In\_Solid in this study.

### 2.2.2 Boundary conditions and loading method

The RC slab was fixed onto the steel legs with shared nodes in the interface, as shown in Fig. 4. To ensure the beam elements and the concrete elements work together, the nodes at the end of longitudinal reinforcements of RC columns were tied to the steel plates by using keyword \*Contact\_Tied\_Nodes\_To\_Surface. Fixed boundary conditions were applied at the bottom of the steel supports. Moreover, eight steel plates were modeled to apply the steel weights (Item 7 in Figure 2) as used in the reference test [20], as shown in Fig. 4. Steel plates were fixed to RC slab with contact surface using keyword \*Contact\_Automatic\_Surface\_To\_Surface (\*CASTS).

As shown in Fig. 5, the load distribution rig from Qian and Li [20] is simulated with high fidelity. It contains a series of rigid beams and plates. The top rigid beams were connected with the secondary rigid beam by revolute joints defined by keyword \*Constrained\_Joint\_Revolute. The secondary rigid beam was connected with the triangle rigid plates by spherical joints defined by keyword \*Constrained\_Joint\_Spherical. Revolute joints were defined between bottom small steel plates and the triangle steel plates to ensure the small plates were able to rotate around the revolute joints. Furthermore, single surface (\*Contact\_Automatic\_Single\_Surface) was defined between the load distribution rig and RC slab.

As shown in Fig. 5, a rigid plate with only vertical freedom was built on the middle of the first rigid beam to apply load on the slab. \*CASTS were defined between the bottom surface of the rigid

180 plate and the top surface of the first rigid beam. The vertical load from the hydraulic jack in the test  
181 program [20] was simulated by applying a velocity-time history for the rigid plate on the middle of  
182 the first rigid beam. The velocity increases from 0 mm/ms under a small constant acceleration at the  
183 beginning to avoid severe vibration of structural resistance, as suggested by the works [10, 28], and  
184 then stays constant. The maximum velocity and acceleration were set to 0.2 mm/ms and  $6.7 \times 10^{-4}$   
185 mm/ms<sup>2</sup>, respectively, by a sensitivity analysis based on Specimen ND.

### 186 2.2.3 Material model

187 In this study, continuous surface cap model (CSCM) was chosen to simulate the concrete  
188 properties, as several studies have proven its accuracy to simulate quasi-static behavior of RC  
189 components subjected to column removal scenarios [10, 14, 28, 29]. The CSCM can effectively  
190 simulate the material properties of concrete (such as damage-based softening and modulus reduction,  
191 shear dilation, shear compaction, confinement effect, and strain rate effect, etc.) under low  
192 confinement situations [30]. Its yield surface consists of shear failure surface and hardening cap  
193 surface [27], as shown in Fig. 6. The original CSCM (\*Mat\_CSCM) requires a series of input  
194 parameters to define concrete material properties. LS-DYNA also provides a simplified CSCM  
195 (\*Mat\_CSCM\_CONCRETE) for concrete properties with unconfined compressive strength between  
196 28 MPa and 48 MPa. The simplified CSCM only needs three input parameters (unconfined  
197 compressive strength  $f_c$ , maximum aggregate size  $A_g$ , and units), and then the remaining material  
198 properties are calculated automatically according to equations proposed by CEB-FIP concrete model  
199 code [31]. The unconfined compressive strength  $f_c$  of Specimens ND and WD was 22.5 MPa and  
200 22.3 MPa, respectively. For simplicity, the average value 22.4 MPa was applied in the numerical  
201 models. The maximum aggregate size  $A_g$  is 10 mm. However, the default concrete material properties  
202 would over-predict the structural resistances of FE models. Therefore, concrete material properties  
203 were made a few adjustments on the fracture energy. Previous studies [10, 28] suggest that the tensile  
204 fracture energy  $G_{ft}$  could take 80 % of the default one when it is over-prediction, and the shear  
205 fracture energy  $G_{fs}$  should be reduced as  $G_{fs} = 50G_{ft}$  (The default is  $G_{fs} = 100G_{ft}$ ) when shear damage is

206 evident. Both  $G_{ft}$  and  $G_{fs}$  are adjusted, as severe punching shear failure occurred in Specimen ND.  
207 The detailed parameters of CSCM are tabulated in Table 2. The unconfined uniaxial stress-strain  
208 relationship of concrete after adjustments is shown in Fig. 7. The CSCM also provides an erosion  
209 algorithm based on maximum principal strain to simulate material failure. If the maximum principal  
210 strain of concrete element is greater than the failure principal strain the concrete element will be  
211 deleted. Although element erosion has little physical meaning, several studies [10, 14, 28, 29] found  
212 that erosion criterion based on the maximum principal strain is a suitable way to simulate concrete  
213 failure under quasi-static condition. However, it will be very hard to simulate the shear failure of  
214 concrete if only the maximum principal strain criterion was used to define the erosion of concrete  
215 element. Thus, the maximum principal strain and shear strain criteria were taken into consideration  
216 in this study by using keyword \*Mat\_Add\_Erosion. Since the appropriate values are dependent on  
217 mesh size, the values of principal strain and shear strain at failure were final set to 0.1 and 0.08,  
218 respectively, by many times of trial calculation based on Specimen ND. Furthermore, the strain rate  
219 effect was ignored since only quasi-static behavior was discussed in this study.

220 The isotropic elastic-plastic material model (\*Mat\_Plastic\_Kinematic) was chosen for  
221 reinforcement. The parameters of material properties, including elastic modulus, yield strength,  
222 tangential modulus, and ultimate strain, were determined in accordance with the material tests. In  
223 addition, the strain rate effect was also ignored.

224 Sensitivity analysis was conducted with three mesh sizes, as listed in Table 3. As shown in Fig.  
225 8, Mesh 2 is reasonable for Specimen ND, as further mesh refinement cannot lead to any remarkable  
226 convergence but instead taking more computing time. Therefore, the mesh size of concrete element  
227 was 18.33 mm×25 mm×25 mm for RC slab and 25 mm×25 mm×25 mm for other components. The  
228 size of beam element was 30 mm.

### 229 2.3 Validation by test results

230 Fig. 9 shows the comparison of load-displacement curves from FE simulation and experimental  
231 tests. For Specimen ND, as shown in Fig. 9(a), after reaching the yield load, the load resistance

decreased due to secondary punching shear failure, which agrees with the experimental observation well. For Specimen WD, as shown in Fig. 9(b), the load resistance decreased slowly after reaching the first peak load (FPL), indicating its failure was mainly controlled by flexural failure, which was similar to that of test results. The error of key results between the FE models and test specimens is less than 10 %, as listed in Table 4. Therefore, the proposed FE models could effectively simulate the behavior of punching shear failure and the effectiveness of drop panels.

It should be noted that the concrete damage was expressed by the damage index. Damage index of 0 and 1 represents no damage and completed failure, respectively. As shown in Figs. 10 and 11, FE model could simulate the crack pattern of tested specimen well. For Specimen ND, FE model could predict the punching shear failure of the slab-column connection well, as shown in Fig. 12. As shown in Fig. 13, the failure mode of Specimen WD could also be well simulated. As a result, the FE models could be used for further parametric study.

### **3. Effects of boundary condition simplification**

For both specimens, due to the limitation of cost and space, only substructures (two-bay by two-bay) were tested. However, in reality, the remaining parts of the building (such as the surrounding slabs and upper floors) may affect the response of the substructures significantly, which was ignored in experimental program. As a result, in this section, the validated FE models were utilized to quantify the effects of boundary condition simplification.

#### *3.1. Effects of surrounding slabs*

Around the substructure, the surrounding slabs will provide certain constraints (rotational, horizontal, or vertical constraints). However, for Specimens WD and ND, the constraints from surrounding slabs were simulated by applying service pressure at the overhang, which is one-quarter of column spacing. Previous works [25] found that the simplified boundary may underestimate the constraints from surrounding slabs. Thus, to further understand the discrepancy between the simplified boundaries and realistic boundaries, four numerical models with different constraints at the overhang (refer to Fig. 14) and one numerical model with four-bay by four-bay (refer to Fig. 15)

258 were developed based on the validated FE model for Specimen ND. As shown in Fig. 14, ND-P, ND-  
259 H, ND-V, and ND-F represent ND with design gravity loads (live and dead) at the overhang, with  
260 rigid horizontal constraint applied at the overhang edge, with rigid vertical constraint applied at the  
261 overhang edge, full constraints (rotational, horizontal, and vertical constraints) applied at the  
262 overhang edge, respectively. It should be noted that the design gravity load (live and dead) was also  
263 applied at the overhang of ND-H, ND-V, and ND-F. Moreover, as shown in Fig. 15, ND-R was  
264 modelled in four-bay by four-bay to include the effects of surrounding slabs realistically. Similarly,  
265 for ND-R, the design gravity load (live and dead) was also applied at the surrounding slabs.

266 Fig. 16 compares the load-displacement curves of ND with varying constraints. The load  
267 resistances of ND-P and ND-V are exactly same but lower than that of ND-R, indicating that the  
268 simplified boundary condition for tested specimens underestimates the constraints from surrounding  
269 bay. In addition, the rigid vertical constraint at the overhang edge has little effects on the load  
270 resistance. Conversely, ND-F with full constraints at the overhang edge (model ND-F) achieves  
271 higher load resistance than that of ND-R. Similar conclusion was obtained by Peng et al. [32].  
272 However, the load resistance of model ND-H is very close to that of ND-R. The comparison of ND-H  
273 and ND-F indicates that the rotational restraint at the overhang edge could further increase the load  
274 resistance. Thus, to achieve more realistic structural response, only rigid horizontal constraints should  
275 be applied at the slab edge.

### 276 3.2. Effects of upper floors

277 Only single-story flat slab substructures were tested in the test program [20]. However,  
278 progressive collapse is a global behavior. Thus, it is necessary to investigate the response of multi-  
279 story flat slab structure. As shown in Fig.17, ND-1F, ND-2F, and ND-3F represent single-story, two-  
280 story, and three-story flat slab substructure, respectively. The dimension, reinforcement details, and  
281 boundary conditions at the overhang edge in each story are identical as those at model ND. The load  
282 distribution rig was generated in each story, and identical service load was applied at the overhangs  
283 of each story.

284 To be consistent, it should be noted that only the load resistance from the first story of these  
285 models was extracted for comparison, as shown in Fig. 18. It can be seen that the load resistance of  
286 ND-2F is extremely similar to that of ND-1F. Although ND-3F achieved largest load resistance  
287 before occurrence of the secondary punching shear failure, the maximum difference is less than 5 %.  
288 As a result, the structural response of the extracted substructure is insensitive to the constraints from  
289 the upper stories.

### 290 *3.3. Combined effects due to surrounding slabs and upper floors*

291 To investigate the combined effect due to surrounding slabs and upper floors, a global FE  
292 model ND-3F-R was built based on ND-3F, as shown in Fig. 19. ND-3F-R (four-bay by four-bay but  
293 three stories) has similar reinforcement details and dimensions as ND-3F and surrounding slabs were  
294 also modelled directly. Design gravity loads (live and dead) were also applied on the surrounding  
295 slabs.

296 Fig. 20 shows the comparison of the resistances of the first story between ND-1F and ND-3F-R.  
297 As can be seen in the figure, the FPL of ND-3F-R is larger than that of ND-1F by 13.8 %. Note that  
298 the FPL of ND-R is larger than that of ND-P by 12.6 % (effect of surrounding slabs while the FPL of  
299 ND-3F is larger than that of ND-1F by 1.0 % (effect of upper floors). Therefore, the effects of  
300 surrounding slabs and upper floors could be superposed.

## 301 **4. Load resisting mechanisms of each story for a multi-storey flat slab structure**

302 As aforementioned, progressive collapse is a global behavior for a multi-storey building.  
303 However, majority of existing tests in progressive collapse investigation were based on single-storey  
304 substructures due to cost and time limitation. These studies are based on the assumption that each  
305 story of the structures has identical load resistance and load resisting mechanisms at same  
306 deformation stages. However, in reality, above assumption is not true even all floors above the lost  
307 column have identical structural components. To evaluate the accuracy of above assumption, the  
308 structural response of each story was extracted for comparison based on the models of ND-2F, ND-  
309 3F, and ND-3F-R.

For ND-2F and ND-3F, as shown in Figs. 21(a) and (b), the structural resistance developing in each story is different. The maximum resistance is observed in the first story. It could be explained that the interaction among different stories, which leads to different in-plane force developed in slab within different stories and mobilization of membrane actions (CMA and TMA). To elaborate on this assumption, the in-plane forces of the slab sections in x direction, as labeled in Fig. 22, were extracted to elucidate the membrane actions developing in flat slab substructure. For simplicity, only the section force of ND-3F was presented. As shown in Fig. 23, the development of the slab-section force in each story is different. In the first story, the section force developing in the slab is in compression (negative) firstly, and then transfers into tension at the large deformation stage. However, the section force of the second story or the third story is always in tension or in compression, respectively. Moreover, the peak value of the section force in the first story is much larger than the ones of other stories. Therefore, for ND-2F and ND-3F, CAA and TMA could develop in the first story effectively, leading to larger resistance. For ND-3F-R, which has a close-to-reality boundary condition, the structural resistance developing in each story is also different. As shown in Fig. 21(c), when the vertical displacement of the middle column is less than 88 mm, the load resistance of the first story is larger than that from the second and third stories.

## **5. Parametric study**

To deeply understand the behavior of flat slab structures to resist progressive collapse, a parametric study was also performed based on the validated FE models.

### *5.1. Effects of integrity reinforcement*

As mentioned above, in the reference tests [20], 3R6 integrity reinforcements were designed passing through the column cages in each principal direction, which was greater than that suggested by ACI 318-11 (2011) [26]. Nevertheless, there was no specific calculation formula for designing of the integrity reinforcement. The arrangement of the integrity reinforcement may affect the load resisting capacity after punching shear failure and deformation capacity of the flat slab structure. Thus, in this section, to quantify the effects of the amount of integrity reinforcement, FE models with

different amounts of integrity reinforcement were simulated. Seven different cases (including 0, 1R6, 2R6, 3R6, 4R6, 5R6, and 6R6 integrity reinforcements) were considered for ND. Moreover, four different cases (including 0, 1R6, 2R6, and 3R6 integrity reinforcements) were considered for ND-R to investigate the effects of integrity reinforcement under a more real boundary condition. To normalize the amount of integrity reinforcement, integrity reinforcement ratio  $\rho_i = A_s/bh_o$  is used.  $A_s$  is the total area of the integrity reinforcements;  $b$  is the column width (200 mm for these specimens); and  $h_o$  is the effective depth of the slab (45 mm for these specimens). Thus,  $\rho_i$  of 1R6, 2R6, 3R6, 4R6, 5R6 and 6R6 are 0.31 %, 0.63 %, 0.94 %, 1.26 %, 1.57 %, and 1.88 %, respectively.

Fig. 24 shows the load-displacement curves of ND with the different amount of integrity reinforcement. The key results are listed in Table 5. As can be seen in the figures and table, with the increase of integrity reinforcement ratio from 0 % to 1.88 % (0 to 6R6), the YL, FPL, and second peak load (SPL) increased by 18.5 %, 41.6 %, and 209.4 %, respectively. It is obvious that increasing the integrity reinforcement ratio can enhance the load resisting capacity at large deformation stage after column removal significantly. This is mainly due to the enhancement of dowel action from integrity reinforcement. However, the efficiency of upgrading the load resisting capacity decreases with increasing the amount of integrity reinforcements. For instance, the YL increases by 15.9 % when the integrity reinforcement ratio increases to 0.94 % (3R6). However, when the integrity reinforcement ratio increases to 1.88 % (6R6), the YL only increases by 18.5 %. Similar phenomenon is observed for the FPL. It is found that when the integrity reinforcement ratio of ND is greater than 0.63 % (2R6), the SPL exceeds the FPL. In summary, to have a good post-punching performance of the flat slab structure subjected to the loss of a middle column scenario, the integrity reinforcement ratio is suggested to greater than 0.63 %.

Comparing to ND, ND-R may be more prone to failure since extra load from surrounding slabs transfers to the adjacent slab-column connections. As shown in Fig. 25, when there is no integrity reinforcement installed passing through the column cages, the structural resistance of ND-R drops to 0 kN suddenly after the secondary punching shear failure occurred. This is because when punching shear failure occurred at one of adjacent slab-column connections, it started to propagate horizontally

363 due to further load redistribution and resulted in total collapse of entire slab, as shown in Fig. 26.  
364 Moreover, similar to ND, it can be found that installing 2R6 ( $\rho_i=0.63\%$ ) integrity reinforcements in  
365 ND-R can ensure that the SPL (180.2 kN) exceeds the FPL (178.5 kN).

## 366 5.2. Effects of slab thickness

367 Previous work [19] investigated different slab thickness of RC flat slab structures subjected to  
368 the loss of a middle column scenario. However, their specimens were tested under concentrated load.  
369 To re-evaluate the effects of slab thickness of RC flat slabs subjected to the loss of a middle column  
370 under UDL loading regime, models with slab thickness of 70 mm and 100 mm were simulated based  
371 on the validated FE models ND and WD. These models are the same as the validated FE models  
372 except the slab thickness.

373 Figs. 27(a) and (b) show the load-displacement curves of WD and ND with different slab  
374 thicknesses, respectively. To distinguish, WD with slab thickness of 55 mm, 70 mm, and 100 mm are  
375 named WD-55, WD-70, and WD-100, respectively. Similarly, ND-55, ND-70 and ND-100 represent  
376 ND with slab thickness of 55 mm, 70 mm, and 100 mm, respectively. As shown in Fig. 27(a), a  
377 thicker slab could increase the load resisting capacity significantly. The FPL of WD-100 is larger  
378 than that of WD-70 and WD-55 by 65.8 % and 177.1 %, respectively, which is different from the  
379 concentrated loading regime in [19]. For WD-70 and WD-100, no obvious re-ascending of load  
380 resistance is observed at the large deformation stage after column removal. This is because the  
381 residual load-resisting capacity at the large deformation stage is mainly provided by the dowel action  
382 from integrity reinforcements and TMA developed in remaining bottom slab reinforcements.  
383 However, increasing the slab thickness has little effects on the development of these actions.  
384 Moreover, by comparing Fig. 13 with Fig. 28, it is found that the failure modes of WD-100 and WD-  
385 70 are quite different to that of WD-55. The main cracks of WD-100 and WD-70 are formed at the  
386 edge of column while those of WD-55 are formed at the edge of drop panels, indicating that the drop  
387 panels of WD-100 and WD-70 lose its efficiency for preventing punching shear failure at slab-

388 column connections. Therefore, the thickness of drop panel should be increased proportionately with  
389 the increase of the slab thickness to ensure its efficiency.

390 For ND-series, similar to WD-series, specimen with a thicker slab has a greater FPL and lower  
391 deformation capacity, as shown in Fig. 27(b). The FPL of ND-100 is larger than that of ND-70 and  
392 ND-50 by 84.0 % and 197.8 %. Moreover, punching shear failure was observed in ND-100 before  
393 reaching its yield load. Conversely, punching shear failure was observed after reaching their yield  
394 load for ND-70 and ND-55, which indicates that the failure mode prone to brittle punching shear  
395 failure with increasing the slab thickness.

## 396 **6. Conclusions**

397 Following conclusions can be made through the studies presented in this paper:

- 398 1. The numerical models built by LS-DYNA are able to simulate the structural behavior of RC flat  
399 slab substructures subjected to a middle column missing scenario under quasi-static loading  
400 regime well. The CSCM employed in the model can effectively predict the punching shear  
401 failure at slab-column connections.
- 402 2. Numerical analysis on different boundary conditions at the overhang edge indicates that using  
403 fixed constraints at the slab edges may over-estimate the response, while only rigid horizontal  
404 constraints at the overhang edge are more realistic.
- 405 3. The numerical results indicated in numerical or experimental studies, only considering first  
406 storey or including upper stories does not alter the response of the first floor greatly. However,  
407 the load resistance from each story in a multi-storey building is different. This is because the  
408 interaction among the stories causes different in-plane force developed in each story, which  
409 influences the mobilization of membrane actions (CMA and TMA).
- 410 4. Increasing the integrity reinforcement ratio can increase the yield load, first peak load, and  
411 second peak load of the specimens, especially for second peak load. The numerical results  
412 indicate that the minimum integrity reinforcement ratio is suggested to be 0.63 % to ensure good  
413 post-punching performance of the flat slab substructure to resist progressive collapse.

5. For RC flat slab structure under uniformly distributed load condition increasing the slab thickness could significantly increase the first peak load while reduce the deformation capacity remarkable. This is because the slab thickness has little effects on the residual load resisting capacity at large deformation stage after column removal.

## 7. Acknowledgements

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496 **Figure caption list**

- 497 **Fig. 1.** Dimension and reinforcement details of Specimen WD (unit: in mm)
- 498 **Fig. 2.** An overview of a specimen in position ready for testing
- 499 **Fig. 3.** The detailing of load distribution rig (Item 9 in Figure 2)
- 500 **Fig. 4.** Numerical model of Specimen WD
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- 502 **Fig. 6.** Yield surface of CSCM model
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522 **Fig. 26.** Failure mode of ND-R without integrity reinforcement  
523 **Fig. 27.** Investigation on effects of slab thickness  
524 **Fig. 28.** Failure modes of WD-70 and WD-100

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**Table 1.** Specimen properties from Qian and Li [20]

Test	Elements				Slab Rebar		
	Interior Column stub	Edge or Corner Columns	Drop Panel Thickness	Drop Panel Rebar, mm	Slab Thickness	Top Layer, mm	Bottom Layer, mm
ND	Height=390 mm Cross-section=200×200 mm <sup>2</sup>	Height=300 mm Cross-section=200×200 mm <sup>2</sup>	N.A	N.A	55 mm	R6@250	R6@250
WD	Reinforcement ratio=1.3%	Reinforcement ratio=2.6%	35 mm	R6@80	55 mm	R6@250	R6@250

**Table 2.** User-input parameters of CSCM (Units: N, mm and ms)

MID 1	RO 0.00232	NPLOT 1	INCRE 0.0	IRATE 0	ERODE 1.10	RECOV 0.0	ITRETRC 0
PRED 0							
G 10396.30	K 11386.43	ALPHA 13.2996	THETA 0.2734	LAMDA 10.5	BETA 0.01929	NH 0	CH 0
ALPHA1 0.74735	THETA1 0.001327	LAMDA1 0.17	BETA1 0.07680	ALPHA2 0.66	THETA2 0.001596	LAMDA2 0.16	BETA2 0.07680
R 5.0	XD 87.6	W 0.05	D1 2.5e-04	D2 3.492e-07			
B 100.0	GFC 3.7760	D 0.1	GFT 0.03776	GFS 0.01888	PWRC 5.0	PWRT 1.0	PMOD 0.0
ETA0C 0	NC 0	ETAOT 0	NT 0	OVERC 0	OVERT 0	SRATE 0	REPOW 0

**Table 3.** Sensitivity analysis on mesh size

Type	Mesh 1	Mesh 2	Mesh 3
Mesh size at flat slab (mm)	30 × 30 × 27.5	25 × 25 × 18.33	15 × 15 × 13.75
Mesh size at other parts (mm)	30 × 30 × 30	25 × 25 × 25	15 × 15 × 15
Length of beam element (mm)	30	30	15
Total number of solid elements	63,891	104,784	392,800
Total number of beam elements	9386	9386	21,550
Computing time (s)	8912	12,152	29,250

**Table 4.** Comparison of the key results between test specimens and FE models

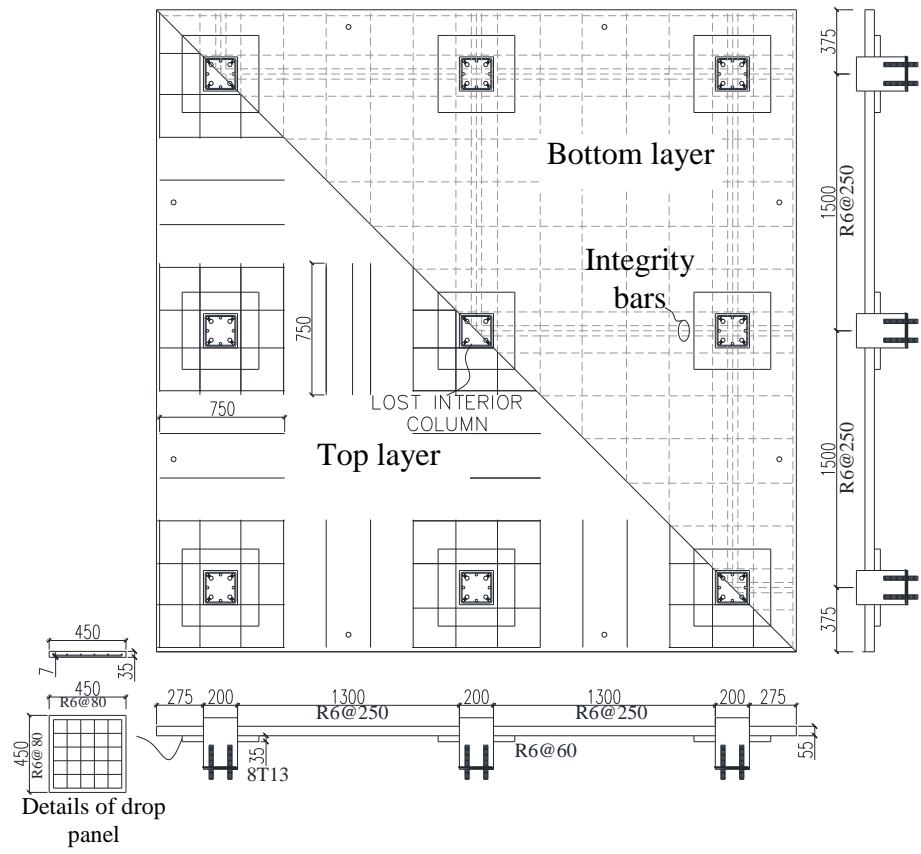
Results Source	ND			WD		
	YL (kN)	FPL (kN)	ULC (kN)	YL (kN)	FPL (kN)	ULC (kN)
Test	134.3	180.8	206.3	185.6	241.5	251.3
FE	137.4	179.5	209.2	190.5	238.6	235.4
FE/Test	1.02	0.99	1.01	1.03	0.99	0.94

Note: YL represents yield load; FPL represents first peak load; ULC represents ultimate load capacity.

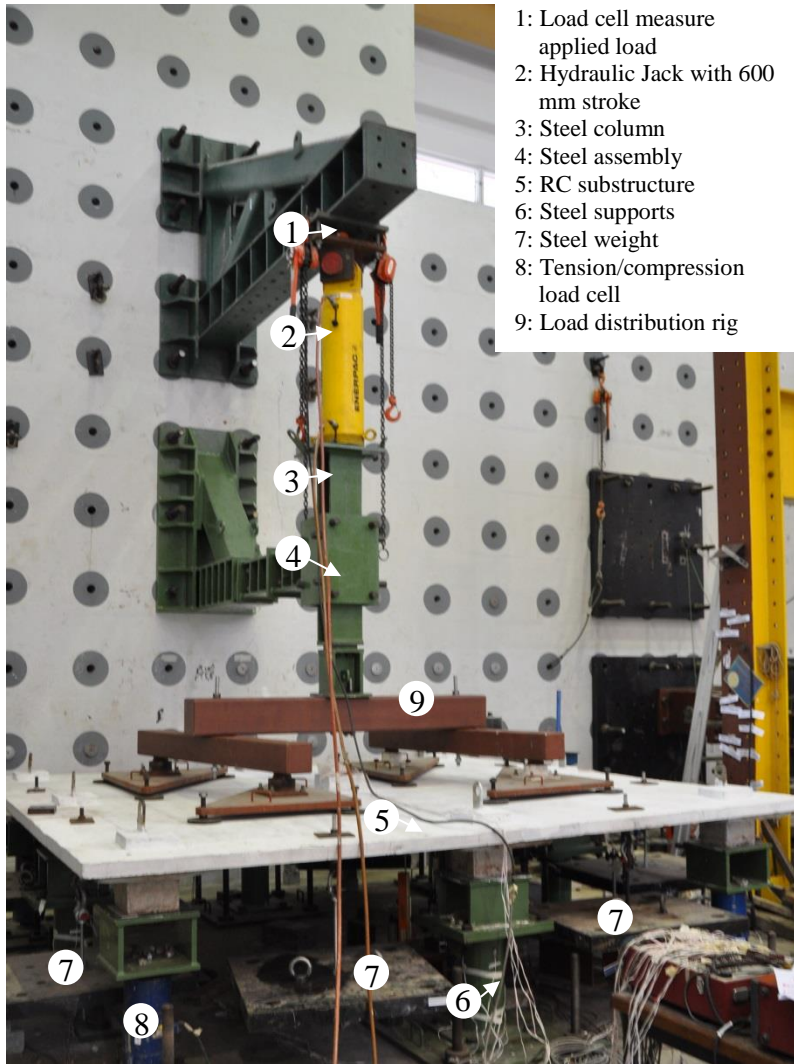
**Table 5.** Key results of ND with different amount of integrity reinforcement

Amount ( $\rho_i$ %)	YL kN	FPL kN	SPL kN	Disp. of SPL mm
None	118.5	147.9	82.3	104.1
1R6 (0.31)	122.6	154.6	123.9	164.1
2R6 (0.63)	126.8	171.7	179.8	154.0
3R6 (0.94)	137.4	179.5	209.2	140.4
4R6 (1.26)	138.2	191.7	224.1	139.9
5R6 (1.57)	138.8	196.9	240.3	123.2
6R6 (1.88)	140.4	209.4	254.6	115.3

Note: YL represents yield load; FPL represents first peak load; SPL represents second peak load.

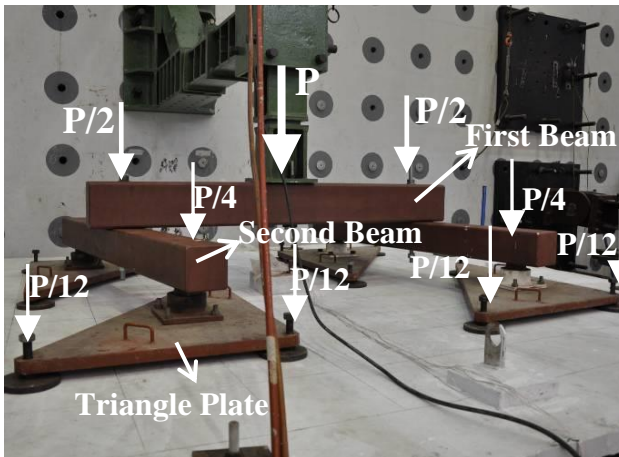


**Fig. 1.** Dimension and reinforcement details of Specimen WD (unit: in mm)

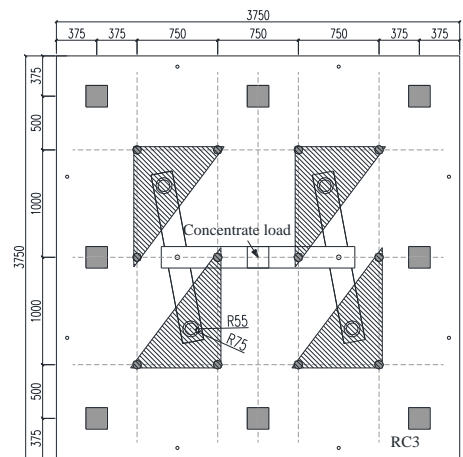


- 1: Load cell measure applied load
- 2: Hydraulic Jack with 600 mm stroke
- 3: Steel column
- 4: Steel assembly
- 5: RC substructure
- 6: Steel supports
- 7: Steel weight
- 8: Tension/compression load cell
- 9: Load distribution rig

**Fig. 2.** An overview of a specimen in position ready for testing

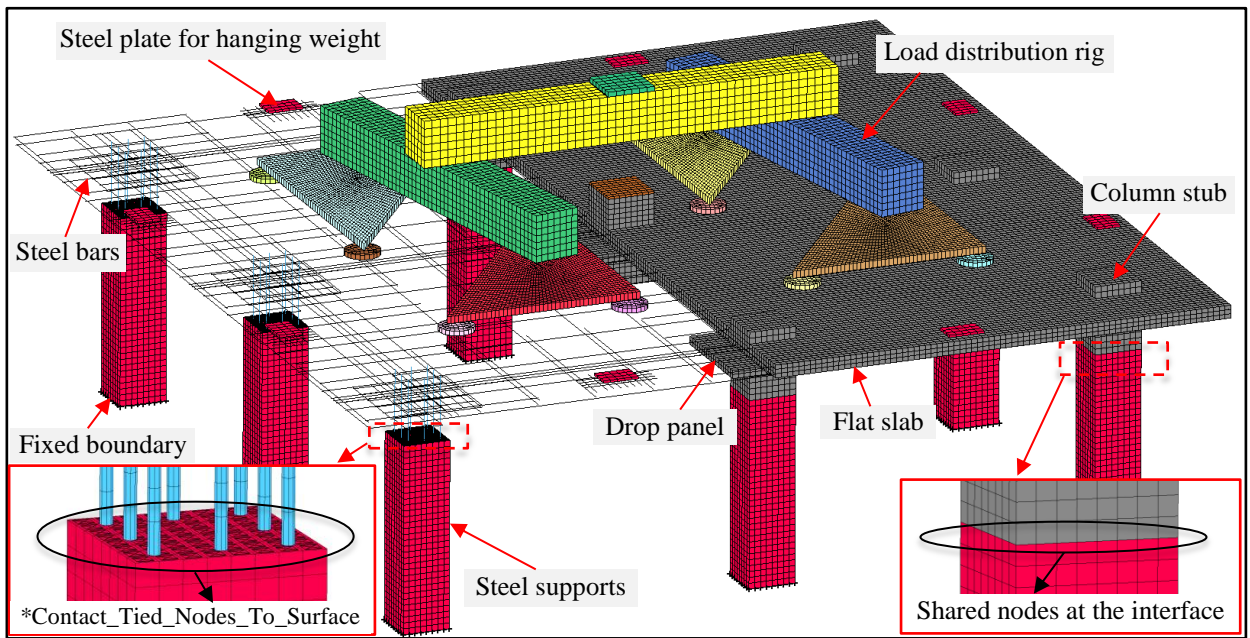


(a) Schematic of the load distribution

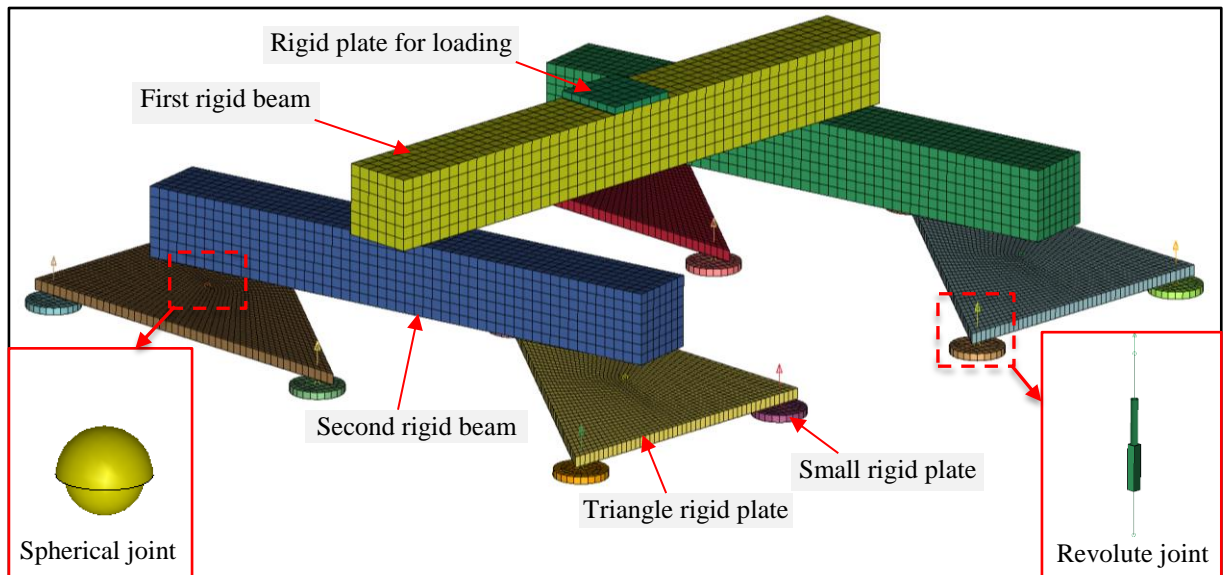


(b) Arrangement of the twelve loading points

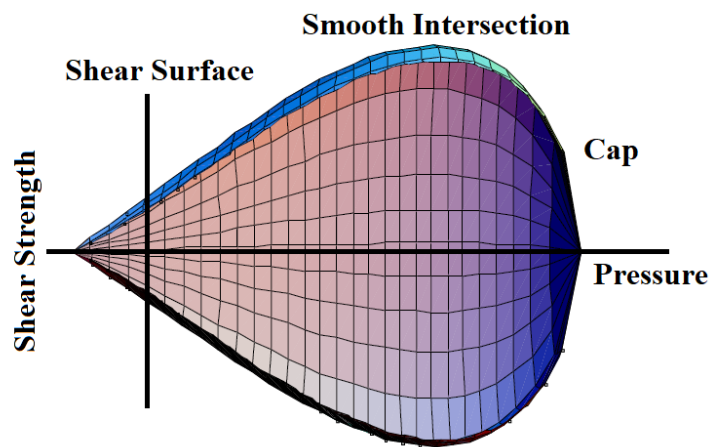
**Fig. 3.** The detailing of load distribution rig (Item 9 in Figure 2)



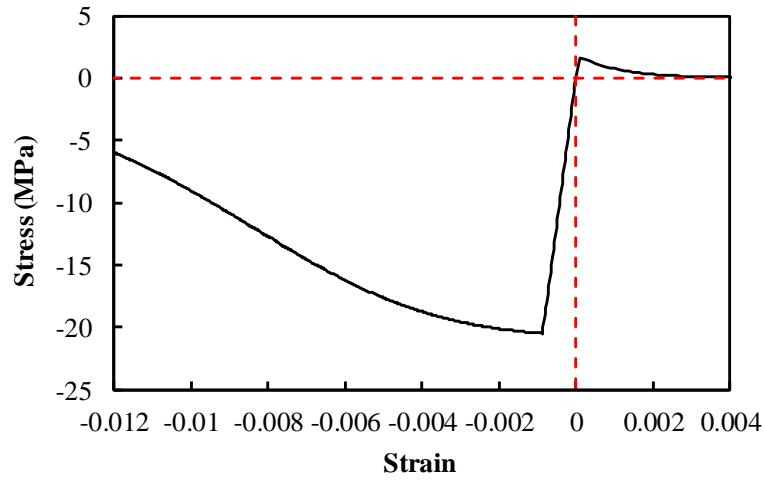
**Fig. 4.** Numerical model of Specimen WD



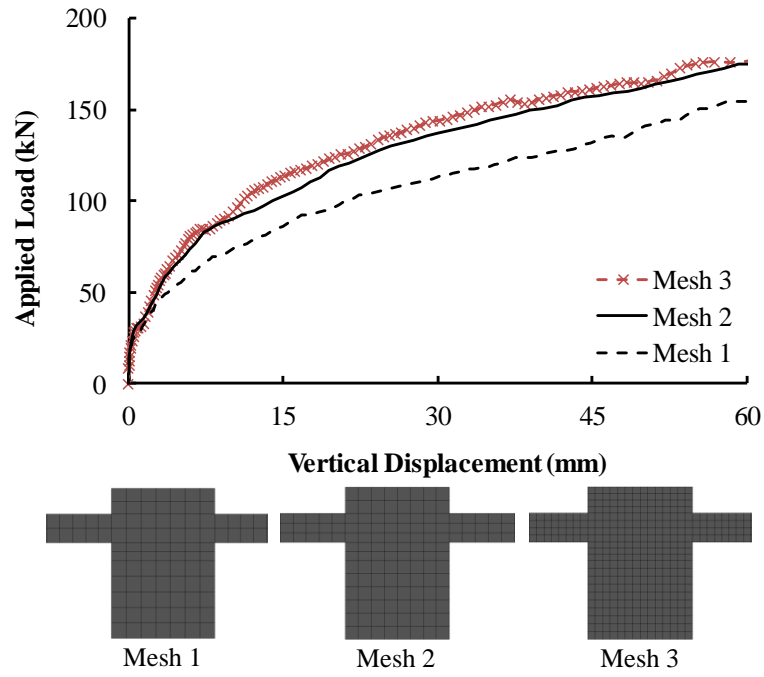
**Fig. 5.** Details of numerical model for load distribution rig



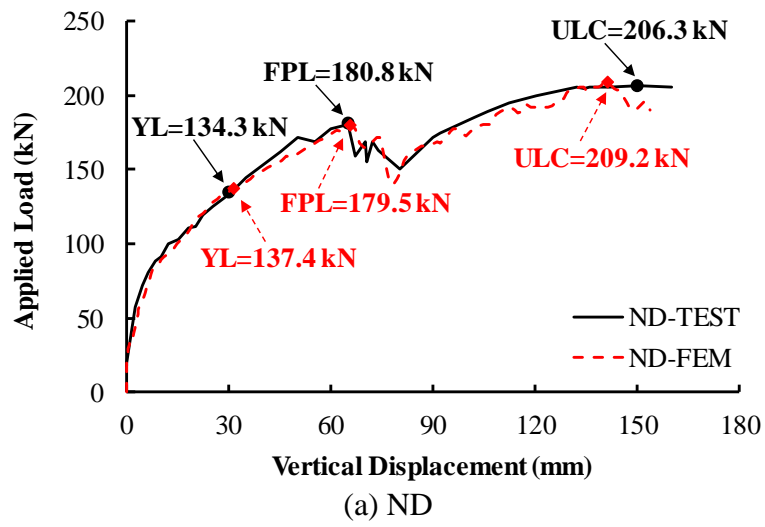
**Fig. 6.** Yield surface of CSCM model

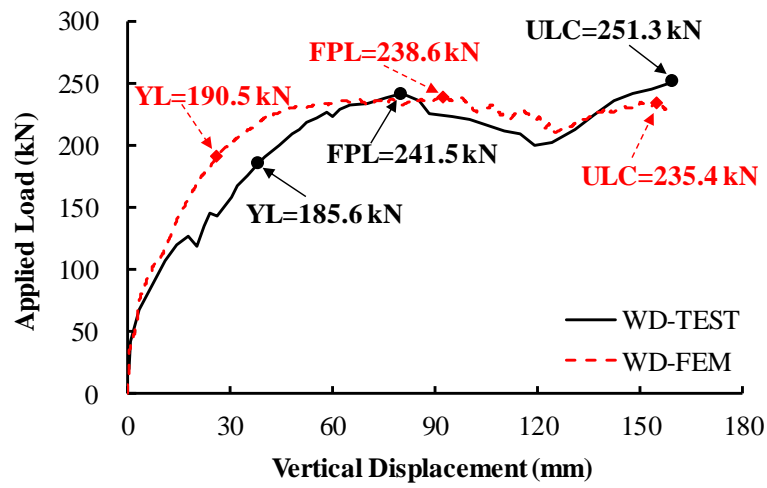


**Fig. 7.** Unconfined uniaxial stress-strain relationship of concrete based on CSCM model



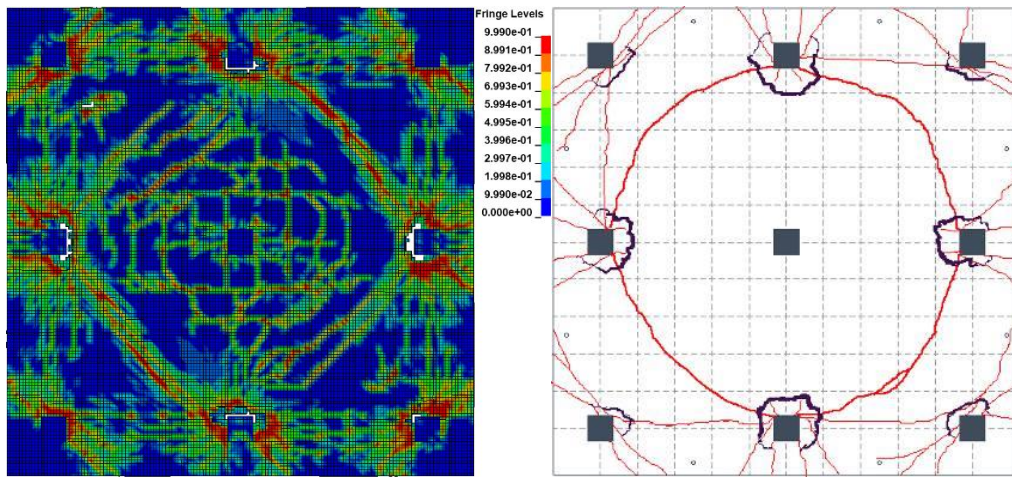
**Fig. 8.** Comparison of different mesh sizes



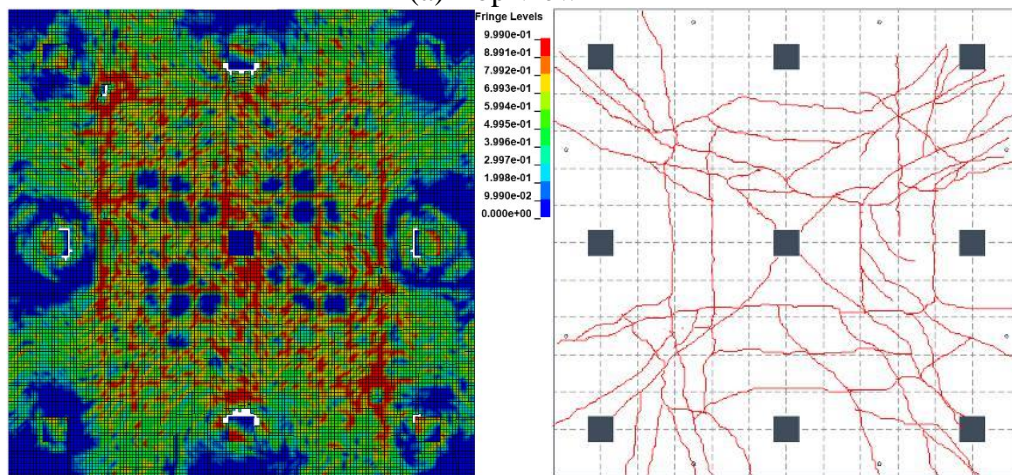


(b) WD

**Fig. 9.** Comparison of the load-displacement curves between simulation and test

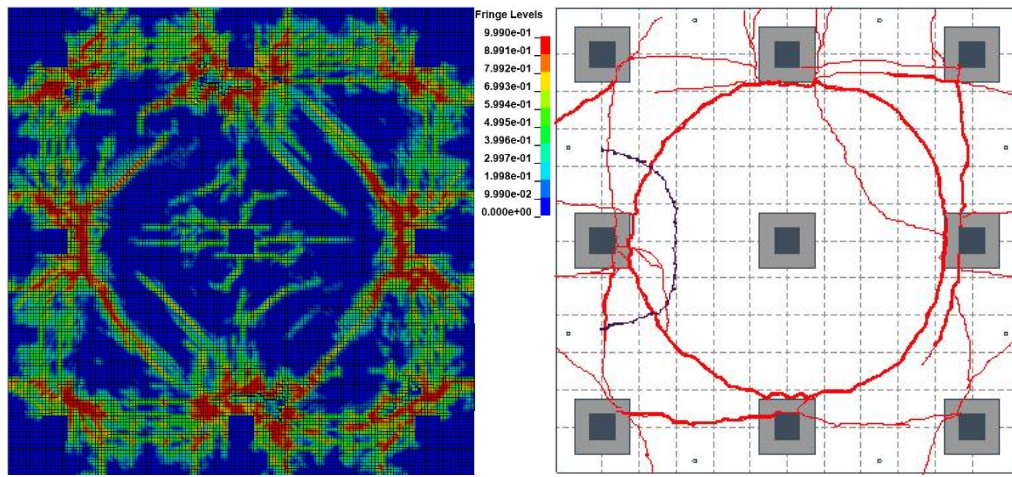


(a) Top view

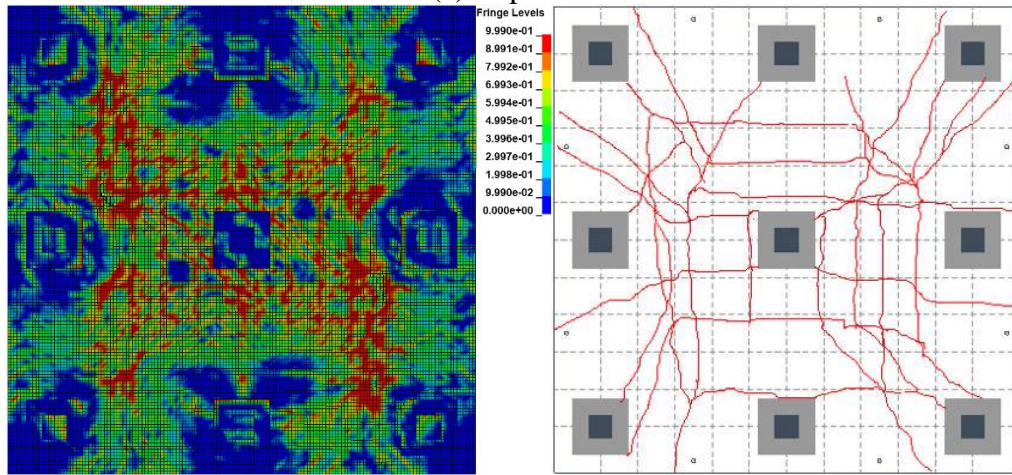


(b) Bottom view

**Fig. 10.** Comparison of crack pattern of ND from simulation and test

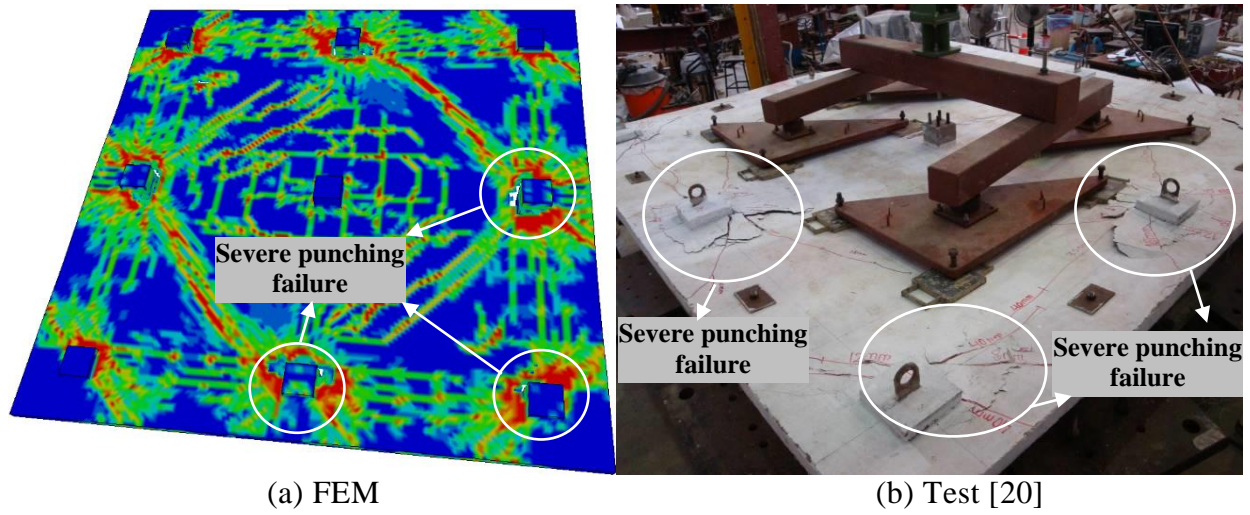


(a) Top view



(b) Bottom view

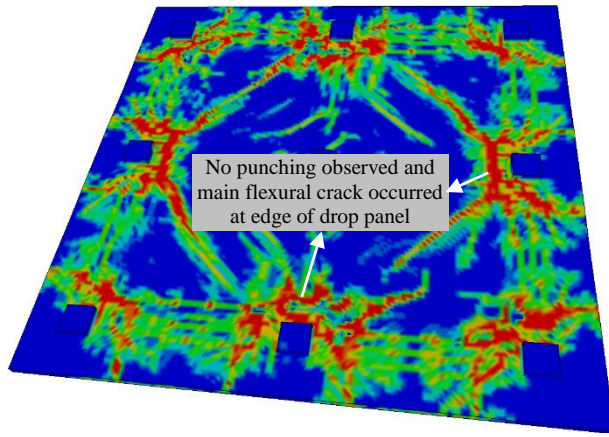
**Fig. 11.** Comparison of crack pattern of WD from simulation and test



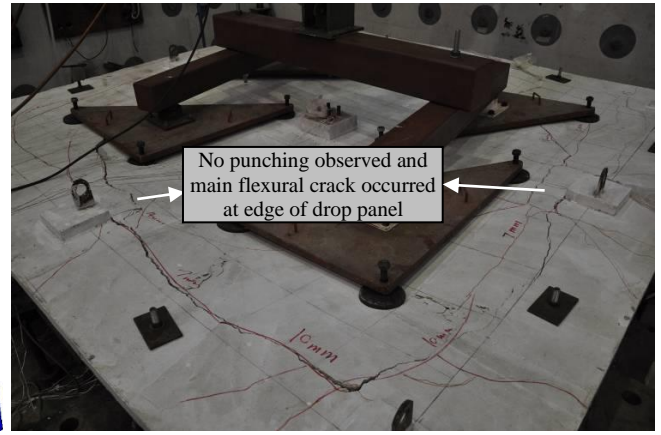
(a) FEM

(b) Test [20]

**Fig. 12.** Failure mode of ND

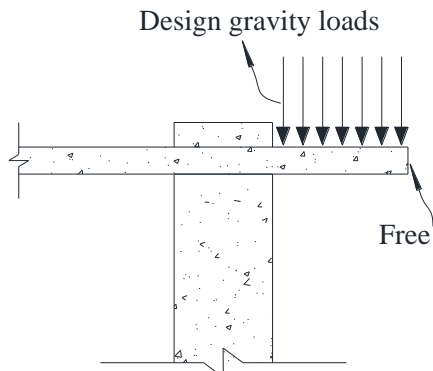


(a) FEM

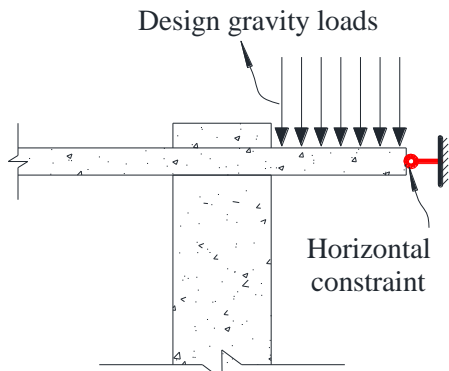


(b) Test [20]

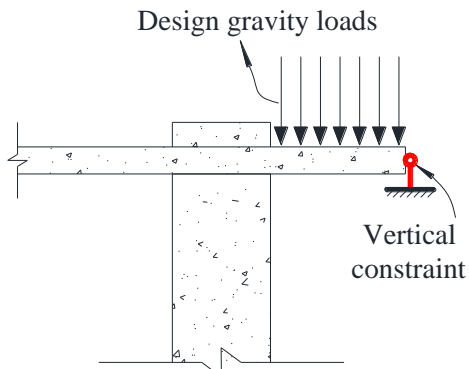
**Fig. 13.** Failure mode of WD



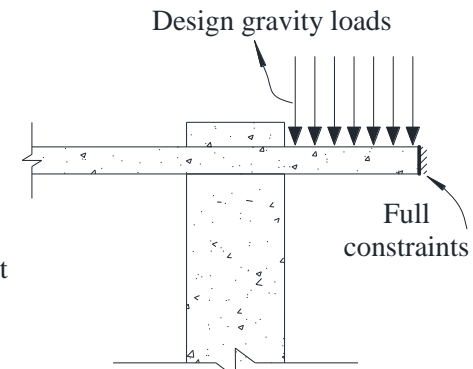
(a) ND-P



(b) ND-H



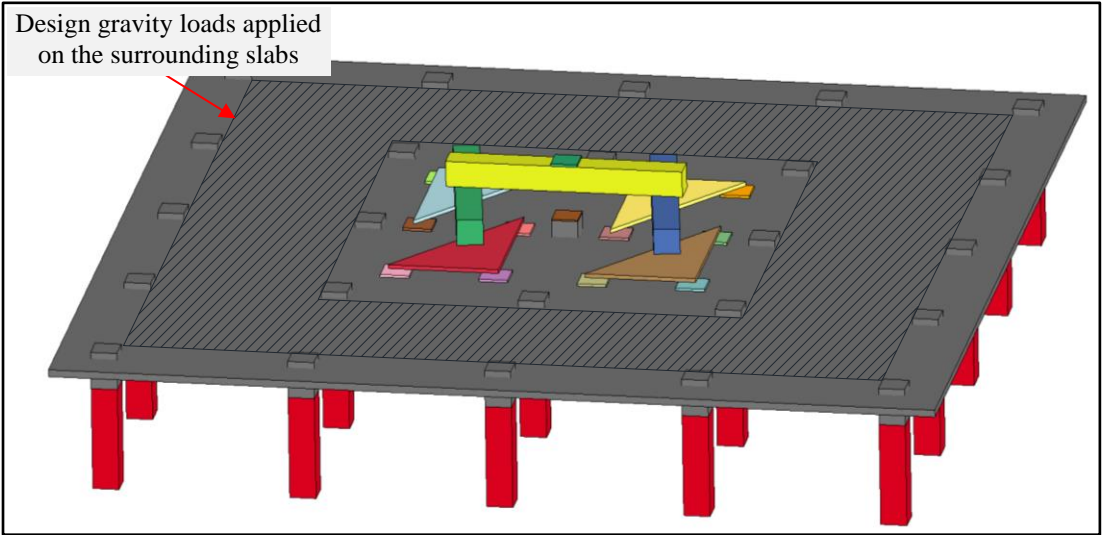
(c) ND-V



(d) ND-F

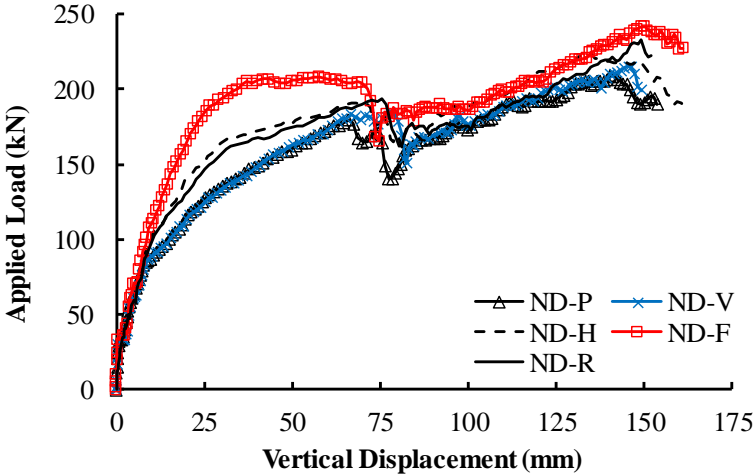
**Fig. 14.** Different constrains of overhang edge

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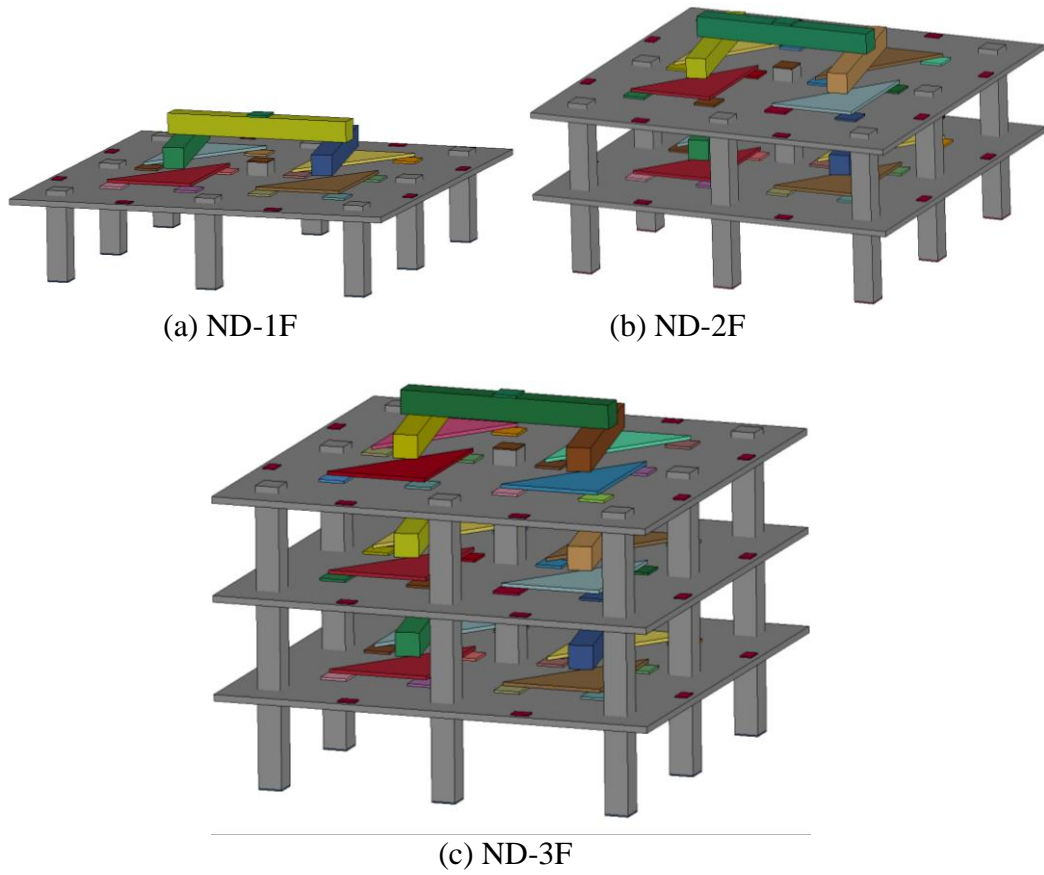
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**Fig. 15.** Numerical model of ND-R

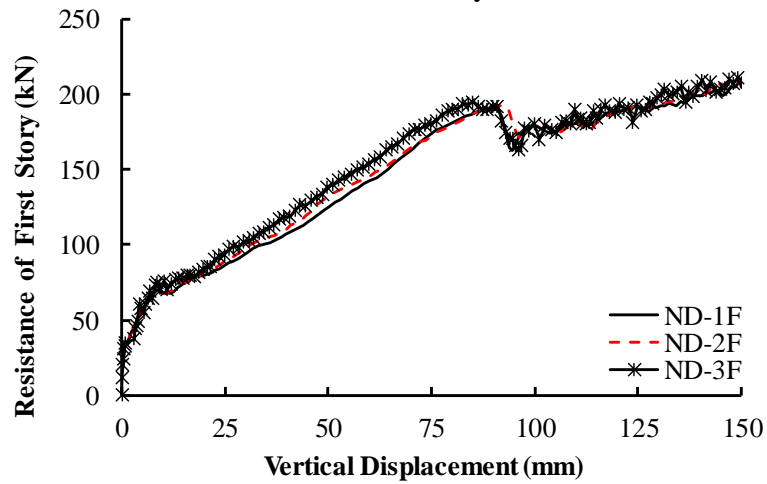


744  
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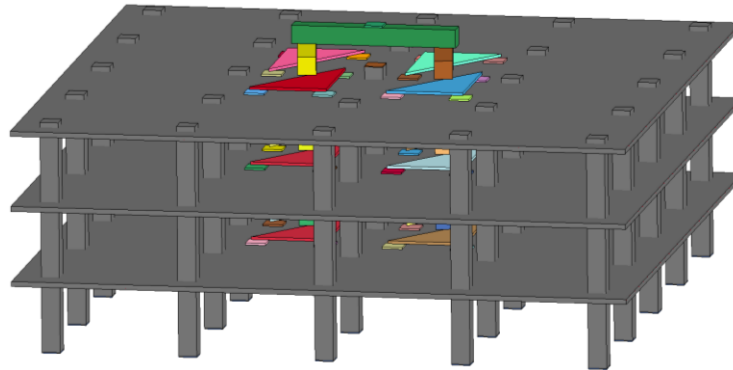
**Fig. 16.** Comparison of load-displacement curve of ND with varying constraints



**Fig. 17.** Numerical models of multi-story RC flat slab substructures

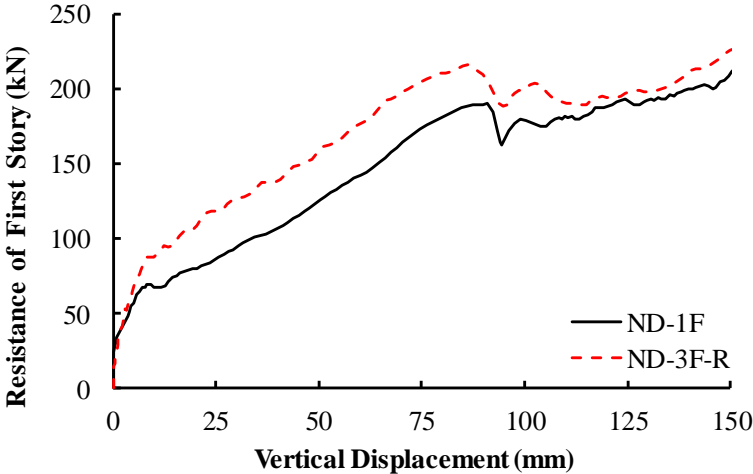


**Fig. 18.** Comparison of resistance in the first story of the multi-story RC flat slab substructures



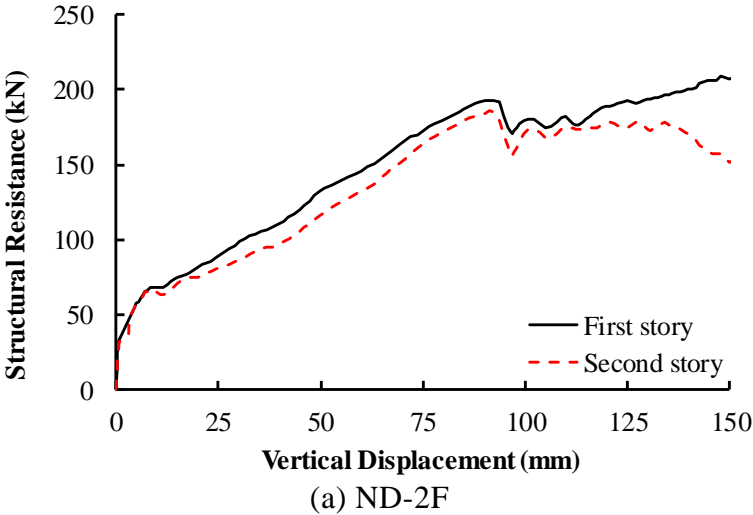
**Fig. 19.** Numerical models of ND-3F-R

769  
770  
771

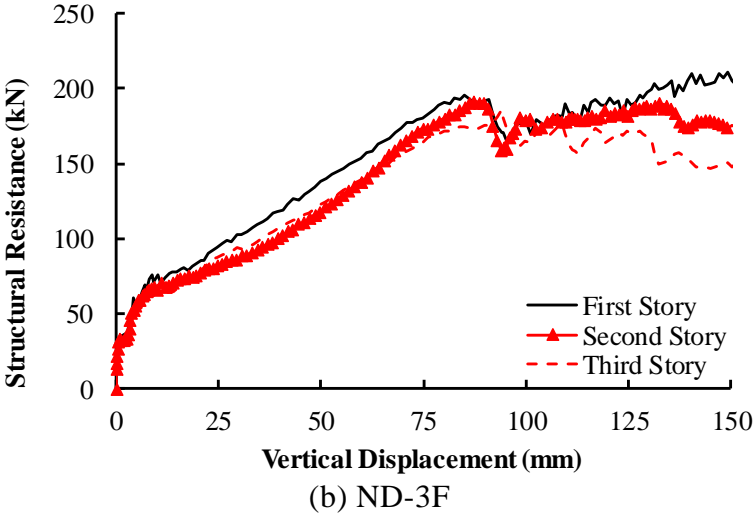


**Fig. 20.** Comparison of the load resistance of the first story from ND-1F and ND-3F-R

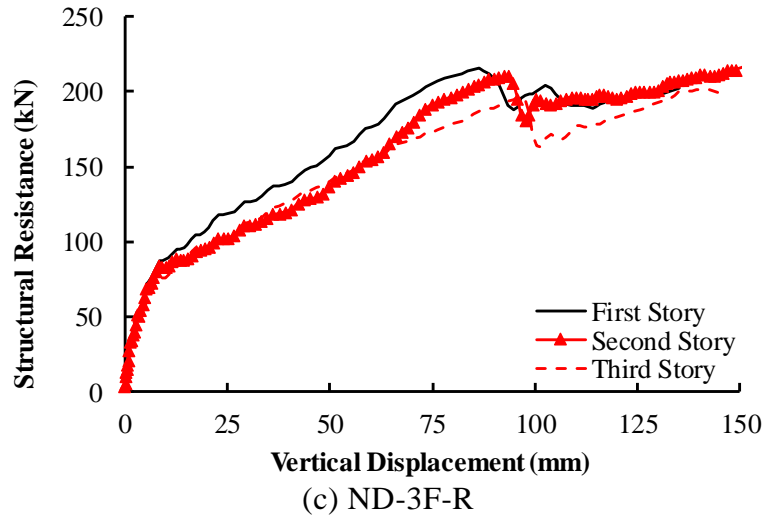
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773  
774  
775



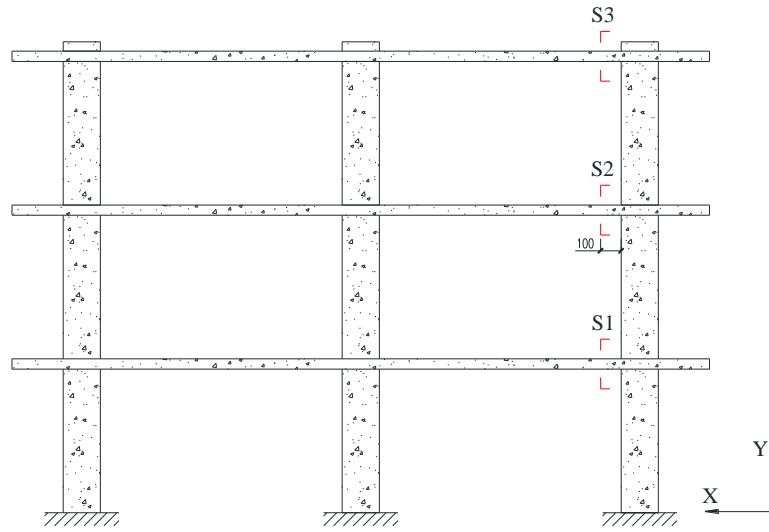
776  
777



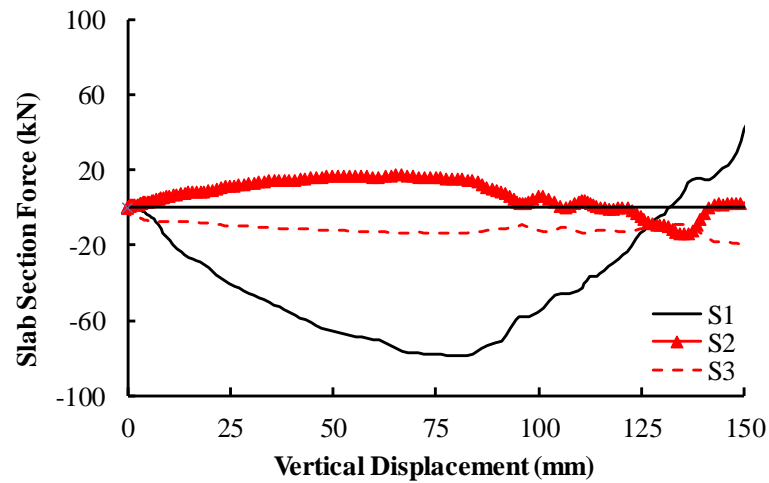
778  
779



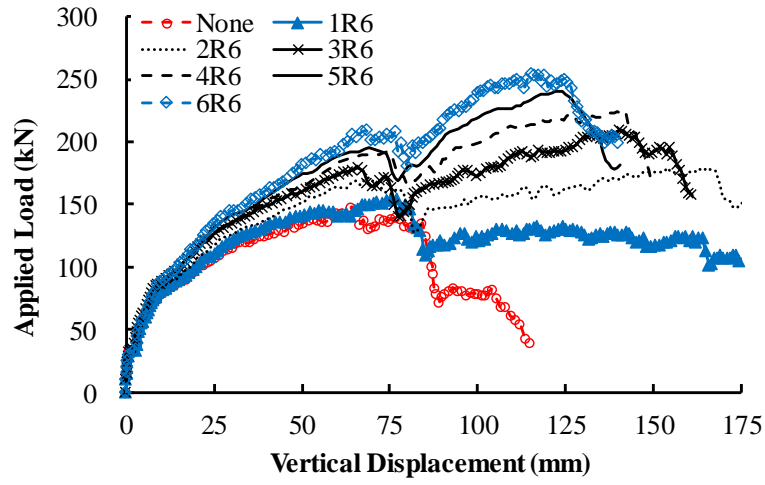
**Fig. 21.** Comparison of the load resistance from different stories



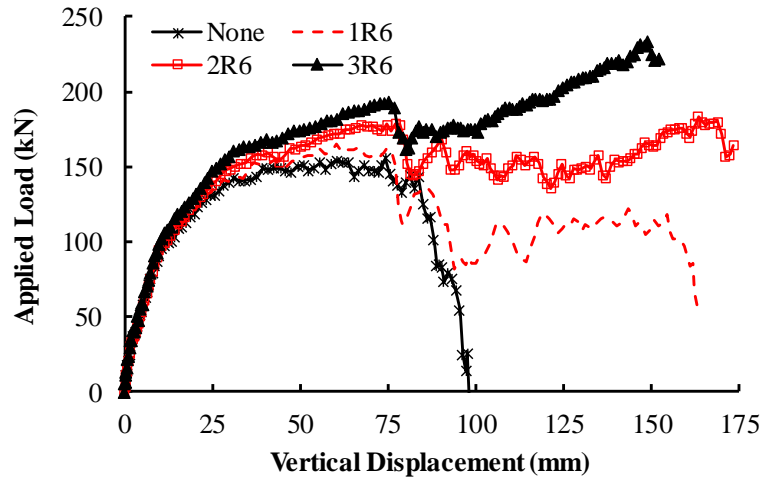
**Fig. 22.** Locations of slab sections



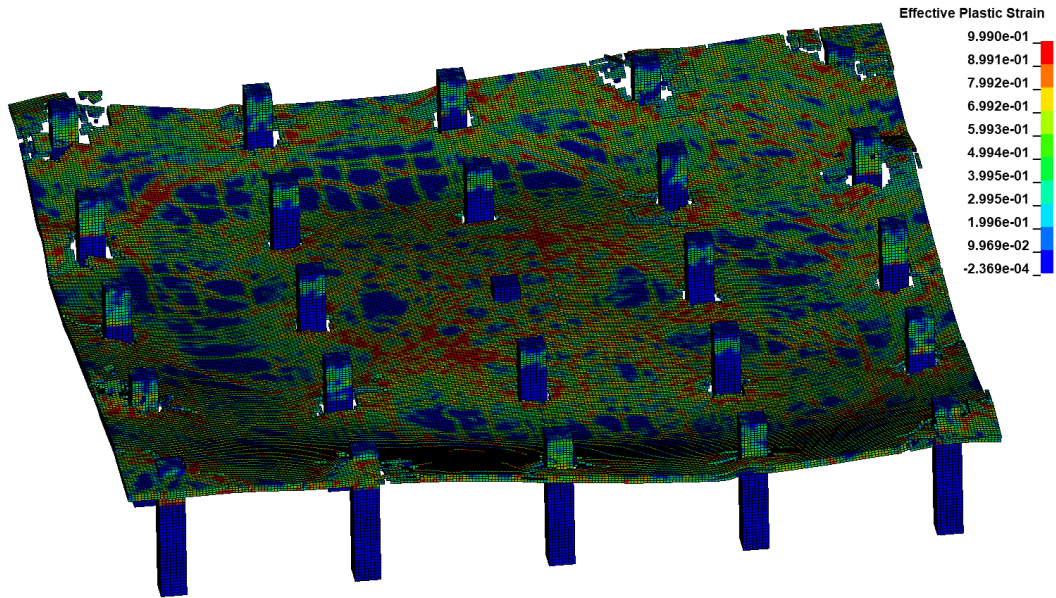
**Fig. 23.** Comparison of the in-plane force in x direction



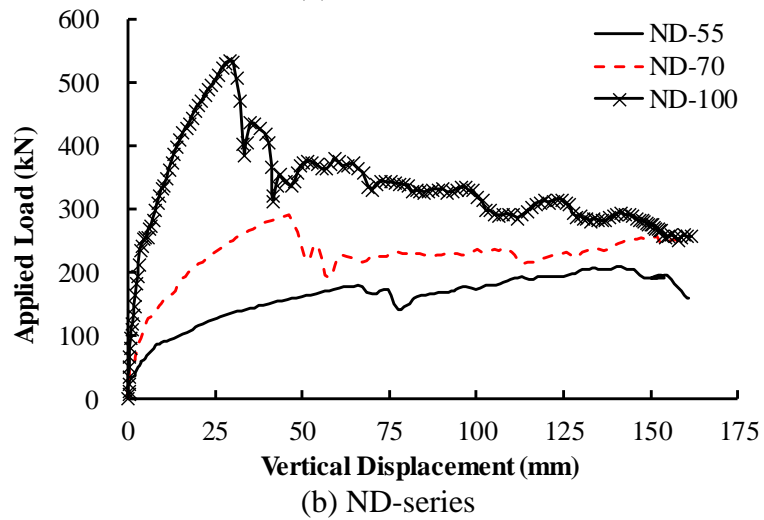
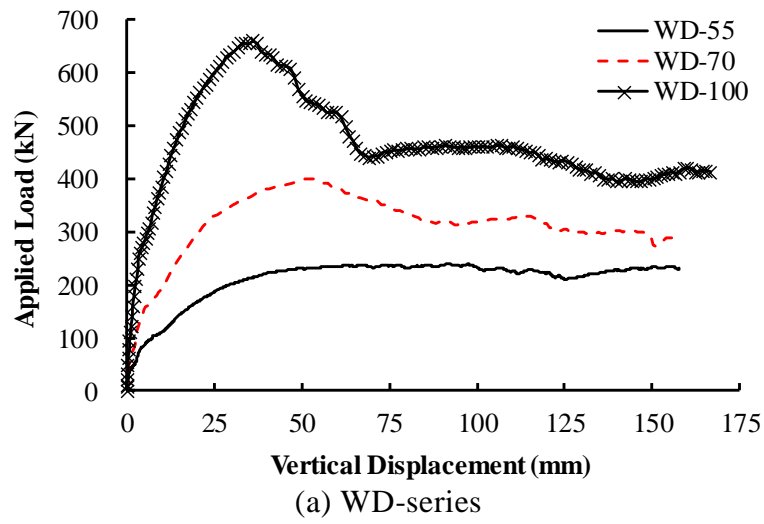
**Fig. 24.** Comparison of ND with different number of integrity reinforcements



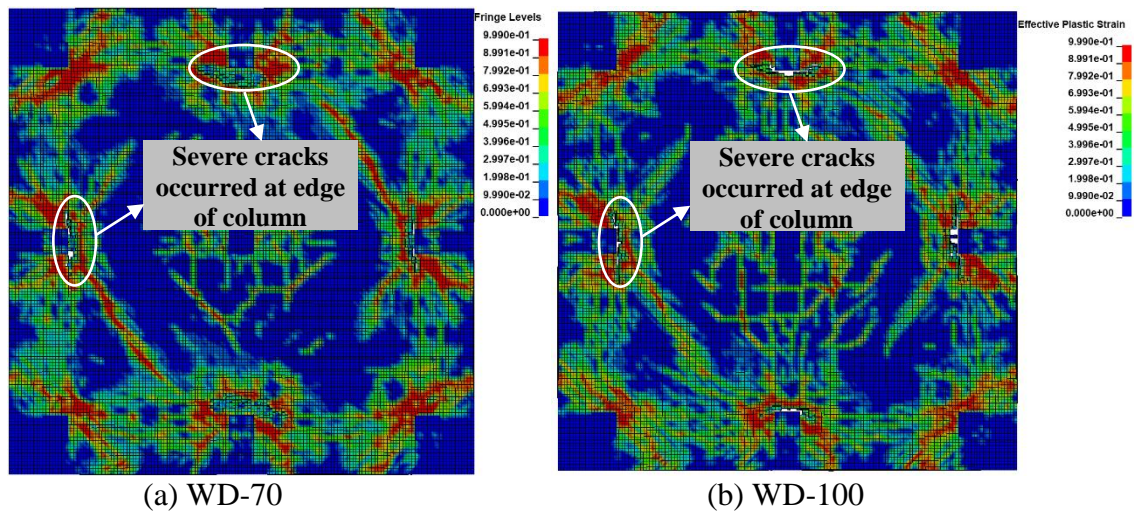
**Fig. 25.** Comparison of ND-R with different number of integrity reinforcements



**Fig. 26.** Failure mode of ND-R without integrity reinforcement



**Fig. 27.** Investigation on effects of slab thickness



**Fig. 28.** Failure modes of WD-70 and WD-100