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<th>Experimental study of drop panel effects on response of reinforced concrete flat slabs after loss of corner column (Experimental and analytical analysis)</th>
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Experimental Study of Drop Panel Effects on Response of Reinforced Concrete Flat Slabs after Loss of Corner Column

Qian Kai and Bing Li

Abstract:
Flat slab structures, with or without drop panels, are popular construction types and have a high occupancy rate. Such flat slab structures are more vulnerable to progressive collapse compared to beam-column-slab structures as there are no beams that could assist in redistributing the load previously carried by the lost column. Therefore, more efforts should be taken to assess the vulnerability of flat slab structures to resist progressive collapse. Unfortunately, few experimental studies have been conducted on this subject to date. Thus, in order to attain a more comprehensive understanding of the behavior of RC flat slabs in resisting progressive collapse and to quantify the influence of the drop panel on the performance of flat slabs against progressive collapse, two series (ND, WD) of one-third scaled specimens were tested under monotonic loading to simulate axial loading in the corner column. The experimental results highlighting the behavior such as force-displacement responses, crack patterns, and failure mechanisms are discussed. Comparison of the performance of these two series of specimens revealed that incorporating drop panels into the flat slabs would increase the first peak resistant capacity by up to 124.7 % and significantly reduce the likelihood of progressive collapse.

Keywords: Progressive Collapse, Flat Slab, Drop Panel, Corner, Reinforced Concrete.

Author Biographical Sketch

Qian Kai SMACI, is a Research Associate in the School of Civil and Environmental Engineering at Nanyang Technological University, Singapore. He received his B.Eng. (Hons) from the Chang’an University, China and MSC from the Nanyang Technological University, Singapore. His research interests are in reinforced concrete structures design, particularly in the area of progressive collapse.

Bing Li MACI, is an Associate Professor in the School of Civil and Environmental Engineering at Nanyang Technological University, Singapore. He received his Ph.D. from the University of Canterbury, New Zealand. His research interests are in reinforced concrete and precast concrete structures, particularly in design for earthquake and blast resistance.
INTRODUCTION

Progressive collapse is defined by the ASCE/SEI 7\textsuperscript{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”. Although progressive collapse is a low-probability phenomenon, the injuries and losses incurred in the event that it takes place could be very severe.

Design guidelines (DoD\textsuperscript{2} and GSA\textsuperscript{3}) have proposed design procedures to evaluate the likelihood of progressive collapse of a structure following the notional removal of vertical load-bearing elements (columns and walls). Resistance to progressive collapse is achieved either implicitly by provisions of minimum levels of strength, continuity, and ductility; or explicitly by (1) providing alternate load paths so that local damage is absorbed and major collapse is averted or (2) providing sufficient strength to structural members that are critical to global stability. The alternate load path method is also frequently used to design structures in resisting progressive collapse due to its independence of abnormal loading conditions. According to this approach, if a primary load-bearing element such as column or wall is destructed during an extreme loading event, an alternate load path must be generated to redistribute the load initially carried by the lost columns or walls.

To study the behavior of RC frame after the removal of one or more columns, several researchers (Marjanishvili and Agnew\textsuperscript{4}, Gurley\textsuperscript{5}) have studied the performance of RC frame following the loss of column via numerical and analytical approaches while Sasani and Sagiorglu\textsuperscript{6}, Yi \textit{et al.}\textsuperscript{7}, Ortan \textit{et al.}\textsuperscript{8}, Su \textit{et al.}\textsuperscript{9}, Yap and Li\textsuperscript{10}, Tian and Su\textsuperscript{11}, and Kai and Li\textsuperscript{12} experimentally investigated the RC frame against progressive collapse. These studies have significantly improved the state-of-the-art standard of protective design and added to the database on progressive collapse behavior of RC structures.

However, majority of the previous tests focused only on beam-column sub-assemblages without including the slabs or beam-column-slab substructures. In typical flat slab or flat plate structures, no beams contributed to the redistribution of the load previously borne by the lost corner column.
Thus, they have higher vulnerability of progressive collapse compared to the beam-column-slab structures. Moreover, flat slab and flat plate structures are popular structure types and have a high occupancy rate. Hence, it is important to determine the extent of vulnerability of these structures in the event of column removal (Ellingwood et al.\textsuperscript{13}). Unfortunately, few experimental studies have been conducted on this subject to date. Thus, in order to attain a more comprehensive understanding of the behavior of RC flat slabs in resisting progressive collapse and to quantify the influence of the drop panel on the performance of flat slabs against progressive collapse, two series of RC flat slabs, referred to as the ‘ND’ (flat slab without drop panel) and ‘WD’ (flat slab with drop panel) series, were tested at NTU, Singapore. By comparing the failure mechanisms and load-displacement responses of these two series of specimens, the effects of the drop panel on the performance of RC flat slab structures can be determined.

**RESEARCH SIGNIFICANCE**

The performance of RC flat slabs with or without drop panels in resisting progressive collapse caused by the loss of a corner column was evaluated in the present study. The primary objective of this paper is to study the drop panel effects on the vertical load-displacement relationship, crack pattern and failure mechanism of the flat slabs by comparing the test results. This study can help structural engineers gain a further understanding of the resistance mechanism of flat slab structures against progressive collapse and provide evidence for the validation of existing numerical modeling approaches.

**EXPERIMENTAL PROGRAM**

**Design of test setup**

It is well known that progressive collapse events are dynamic phenomena. In-situ tests represent the preferred method to study the behavior of RC flat slab structures for progressive collapse. However, the tremendous costs of the in-situ tests mean that it is impossible to systematically investigate the behavior of RC flat slab structures against progressive collapse via this method. The
experimental results of Sasani and Sagiroglu\textsuperscript{6} and Yi \textit{et al.}\textsuperscript{7} indicated that the upper and lower floors operate in tandem as a unit as long as the dimensions and reinforcement details in each floor are similar. Thus, the behavior of a multi-storey frame could be simplified to that of a single-storey substructure with proper boundary conditions. Fig. 1 demonstrates the deformation shape of a nine-storey flat slab structure after one of the ground corner columns was lost. As shown in the figure, the deformation was concentrated in the corner panels. Therefore, one typical critical panel (corner panel in second storey) was extracted and studied. A schematic of the test setup is shown in Fig. 2. Three rigid steel legs were utilized to support the slab and each steel leg was connected with a 75.0 mm (2.95 in.) thick strong plate through four $\Phi27$ bolts. The steel plates were fastened to the strong floor using pre-tensioned steel rods. Although major deformation was concentrated in the corner panel and limited deformation was observed in the adjacent bays following the removal of a corner column, the continuity of the slab could provide additional constraints on the slab and affect the realism of the test results. In order to partially simulate the influence of the continuity of the slabs on the overall performance, the slab was extended beyond the fixed-support by one-fourth of the span in both directions. Five steel weight assemblies were applied on the extended part of the slab to simulate the influence of the continuity of surrounding slabs on the response of the specimens. It should be noted that the weight of the steel assemblies was determined via assuming the design service pressure was applied on the extended parts. The existing axial load in the corner column before it was lost was simulated by applying downward displacements at the corner column stub through a hydraulic jack with a 600.0 mm (23.62 in.) stroke.

Sasani and Sagiroglu\textsuperscript{6} identified three-dimensional (3-D) Vierendeel action as the major mechanism for the redistribution of loads in the framed structures under the scenario of the loss of a corner column. However, there no related in-situ tests conducted for flat slab structures to date. Thus, whether the Vierendeel action still dominates the load redistribution for a flat slab in the case of the loss of a corner column has not been fully understood. Thus, the numerical approaches
(ABAQUS 6.9\textsuperscript{14}) were utilized for this purpose. The numerical model was initially validated by the results obtained from Sasani \textit{et al.}\textsuperscript{15} and the details of the numerical procedure can be found in Kai\textsuperscript{16}. The numerical results (as shown in Fig. 1) indicated that the column strips connected with the corner column deformed in double curvature after the removal of the corner column. Moreover, significant positive bending moment (tension at the bottom of the slab) was observed in the column strip local to the corner column after the removal of the corner column. Therefore, it can be concluded that Vierendeel action still contributed to the load redistribution for flat slabs. Thus, in order to simulate the effect of the Vierendeel action applied on the test specimen (Kai and Li\textsuperscript{12}), the rotation of the corner column should be partially constrained. Fig. 3 illustrates the details of the steel assembly utilized to apply the Vierendeel action equivalently to the tested specimens. One strong steel column was connected to the corner stub of the RC specimen using pre-cast bolts. Four high strength and stiffness steel pins were used to apply prescribed partial rotational and horizontal constraints in each direction. In other words, the steel column could move freely in the vertical direction but its rotational and horizontal freedoms were partially restrained. However, it should be pointed out that the extent of rotational constraint on the corner column for flat slabs should be different compared to that for beam-column-slab structures and the extent of this difference should be evaluated.

For test specimens, the numerical models (ABAQUS 6.9\textsuperscript{14}) indicated that the center of the joint just above the lost column has a maximum outward horizontal movement about 4.2 mm (0.17 in.) whereas the vertical displacement (D\textsubscript{1}) is about 180.0 mm (7.09 in.). However, it should be noted that the rotational constraint assembly utilized in Kai and Li\textsuperscript{12} was designed in accordance with the outward horizontal movement being 7.2 mm (0.28 in.) when the vertical displacement (D\textsubscript{1}) is about 180.0 mm (7.09 in.). Thus, the test results presented in the present tests are slightly conservative as the same rotational constraint assembly utilized in Kai and Li\textsuperscript{12} was also used here. The allowance between the steel pin and the hole was designed as follows:
\[
\phi = \frac{H_1}{TV} = \frac{H_1}{V + D_1} = \frac{7.2}{625 + 180} = 8.9 \times 10^{-3}
\] (1)

where \( \phi \) is the design rotation of the steel column; \( H_1 \) is the horizontal movement of the center of the joint just above the damaged column; \( TV \) is the total vertical distance between the center of the steel box to the center of the corner joint when the specimen has vertical displacement of \( D_1 \); \( V \) is the vertical distance between the center of the steel box to the center of the corner joint at beginning of the test; and \( D_1 \) is the vertical displacement.

\[
\delta = \frac{\bar{V} \times \phi}{2} = \frac{350 \times 8.9 \times 10^{-3}}{2} = 1.56
\] (2)

where \( \bar{V} \) is the average vertical distance between two steel pins in each direction; and \( \delta \) is the difference between the diameter of the hole and the steel pin.

Therefore, the diameter of the steel pin was 40.0 mm (1.57 in.) while the diameter of the hole in the steel box was designed to be 43.0 mm (1.69 in.) as shown in Fig. 3.

**Experimental substructures**

Two series of column-slab substructures, referred to as the ‘ND’ series (flat slab without drop panel) and ‘WD’ series (flat slab with drop panels), were constructed to study the effects of varying the amount of slab reinforcement in each series. The additional amount of resistance provided by incorporating the drop panels could be determined by comparison of the test results of these two series of specimens. The dimensions and reinforcement details are summarized in Table 1. As illustrated in the table, the only difference between WD1, WD2 and WD3 is the amount of the slab reinforcement. Fig. 4 demonstrates the dimensions and reinforcement details in WD1. As shown in Fig. 4, the concrete cover of the column and slab are 20.0 mm and 7.0 mm (0.79 in. and 0.28 in), respectively. For the WD-series specimens, there is one corner column stub, three enlarged columns and four drop panels cast monolithically. The corner column stub representing the removed column was a 200.0 mm (7.87 in.) square for all specimens while the enlarged columns were 250.0 mm (9.84 in.) squares to ensure failure would not occur in these enlarged...
columns. Moreover, the reinforcements were installed in both the top and bottom of the slab to prevent possible brittle failure of the specimen within the small deformation stage due to punching shear failure occurring in the column-slab connections. The thickness of the drop panel was 40.0 mm (1.57 in.) and the reinforcement in the drop panel was one layer rebar spaced at 70.0 mm (2.76 in.). Four $\Phi 25$ bolts were pre-cast in each enlarged column and connected with the steel legs. Fig. 5 illustrates the slab reinforcement details of ND2 and WD2. As presented in Fig. 5, the slab reinforcement in the middle strip comprised the R6 rebar at 125.0 mm (4.92 in.) in two layers at the top and bottom whereas the column strip was composed of two layers of R6 rebar spaced at 60.0 mm (2.36 in.) and 125.0 mm (4.92 in.) at the top and bottom, respectively.

Specimens ND1, ND2 and ND3 corresponded to WD1, WD2 and WD3, respectively. As shown in Table 1, similar details were provided in the columns and slabs of the ND-series specimens as the corresponding WD-series specimens while no drop panels were incorporated. High yield strength steels were used for the longitudinal reinforcement (T16) while mild-steel was used for the transverse and slab reinforcements (R6). It should be noted that T16 and R6 represent deformed rebar with diameter of 16 mm and plain rebar with diameter of 6 mm, respectively. The average concrete compressive strengths were about 19.5 MPa ($4.07 \times 10^5$ psf) and 26.0 MPa ($5.43 \times 10^5$ psf) for the ND-series and WD-series specimens, respectively. It should be noted that the prototype WD2 was designed in accordance with ACI-08. The dead load of the prototype structure due to the 210.0 mm (8.26 in.) thick slab was 5.1 kPa (106.6 psf). The additional dead load was assumed to be 1.0 kPa (20.9 psf). The equivalent additional dead load due to the weight of in-fill walls was 2.25 kPa (47.0 psf). The live load was assumed to be 2.0 kPa (41.8 psf). One-third scaled substructures were cast and tested in this study. A uniform pressure of 11.0 kPa (229.7 psf) based on loading combination (1.2DL+0.5LL), which was suggested in DoD guidelines, was applied on the surface of the prototype slabs. In order to create same demand/capacity ratio on the critical slab section of the scaled-down slabs as that of the prototype slabs, same magnitude of the
pressure (11.0 kPa) should be applied on the scaled-down slabs. The design axial force in the
corner column of each specimen as recommended by DoD\textsuperscript{2} guidelines is listed in Table 1.

**Instrumentations**

Extensive measurement devices were installed both internally and externally in order to monitor
the response of the test specimens. A total of 35 and 42 data channels were active during the
testing process for the ND- and WD-series specimens respectively. A load cell was used to
measure the applied force on the corner stub. Two tensile and compressive load cells were
horizontally connected with the steel assembly and were used to measure the horizontal reaction of
the box in each direction. One LVDT with 300.0 mm (11.81 in.) travel was installed vertically to
measure the vertical movement of the corner column stub during the test. In order to monitor the
horizontal movement of the corner joint during the test, a displacement transducer with 1200.0 mm
(47.24 in.) travel was installed horizontally. The remaining six displacement transducers were
placed vertically to monitor the deflection of the slab. For ND-series specimens, a total of 23 strain
gauges were mounted on the reinforcement at strategic locations in order to monitor the strain
variation along the corner column and slab during the test. For WD-series specimens, strain gauges
were not only installed in the column and slab but were also placed in the drop panels. The
locations of the strain gauges placed in the drop panel and corner column are shown in Fig. 4 while
the locations of the strain gauges in the slab reinforcement are illustrated in Fig. 5.

**EXPERIMENTAL RESULTS OF COLUMN-SLAB SUBSTRUCTURES**

Two series (ND and WD) of one-third scale flat slab substructures were constructed and tested to
evaluate the drop panel effects on the performance of flat slabs for progressive collapse caused by
the loss of a ground corner column with different amounts of slab reinforcement. The key points of
the test results of the six specimens are summarized in Table 2 and discussed below.

**Global behavior**

...
ND1—the measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of ND1 with different performance levels is shown in Fig. 6a. Four performance levels at significant parts of the test were identified. PL1, PL2, PL3 and PL4 represented the first flexural crack, the first yield of the slab reinforcement local to the enlarged adjacent column, the first peak capacity and the beginning of the development of tensile membrane action, respectively. The first crack was observed at the interface between the slab and the adjacent enlarged column at a load of 1.8 kN (0.41 kip). Following the first crack, several flexural cracks were observed at the bottom of the slab close to the corner column due to equivalent Vierendeel action. At a load of 6.3 kN (1.42 kip), the first diagonal crack in the slab was formed and passed through the center of the slab. However, the first yield of the top reinforcement was observed at a load of 7.3 kN (1.62 kip) and corresponded to a vertical displacement of 30.9 mm (1.22 in.). When the vertical displacement reached 70.3 mm (2.77 in.), the first peak capacity $P_{cu}$ was attained at a load of 8.5 kN (1.91 kip) and corresponded to 53.4 % of the design axial load as recommended by DoD\textsuperscript{2}. At this load stage, more diagonal cracks parallel to the first diagonal crack were formed and these diagonal cracks were moving toward the corner column. The resistance of the specimen began to decrease with further incremental of the vertical displacement due to severe yield of the slab reinforcement and wider opening of the diagonal cracks. When the displacement reached 120.3 mm (4.74 in.), which was equivalent to 4.2 % of the tip displacement ratio (TDR), defined as the ratio of vertical displacement at the center of the corner stub to column spacing, the load-displacement curve began to ascend again (attributable to tensile membrane action). A diagonal crack penetrated through the depth of the slab with a further increase in vertical displacement. At the end of the test, punching shear cracks were observed in the corner column-slab connection. It should be noted that no obvious punching failure was observed in the top slab around the adjacent columns. Fig. 9a illustrates the crack pattern development corresponding to different performance levels of ND1. It should be pointed out that no cracks were observed in the corner column and joint region during
the test. This was significantly different from the failure mode of the beam-column-slab substructures tested by Kai and Li\textsuperscript{12}.

Both the transverse and longitudinal horizontal reaction forces were measured by the tension/compression load cells (item 4 in Fig. 2), which were connected with the steel assembly. As shown in Fig. 6a, the recorded horizontal compressive force was limited before the first crack occurred in the specimen. However, it significantly increased after the first crack was observed (similar behavior was observed in Kai and Li\textsuperscript{12}). The recorded response of the horizontal reaction force in the transverse direction was almost identical as that measured in the longitudinal direction. A maximum compressive force of 4.1 kN (0.92 kip) and 4.5 kN (1.01 kip) were measured in the transverse and longitudinal directions at the displacements of 70.3 mm (2.77 in.) and 80.1 mm (3.15 in.), respectively. It should be noted that the measured compressive force did not represent the horizontal axial force developed in the center of the corner joint as majority of the compressive force was utilized to balance the positive bending moment at the slab-corner column connection.

When the displacement reached 202.4 mm (7.97 in.), which was equivalent to 8.4 \% of the TDR, tensile reaction forces were recorded in both horizontal load cells. At the final stage of the test, the maximum horizontal tensile reaction forces measured in the transverse and longitudinal load cells were 6.0 kN (1.35 kip) and 6.1 kN (1.37 kip), respectively.

\textit{WD1}—the measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of WD1 is shown in Fig. 6a. At a load of 3.2 kN (0.72 kip), flexural cracks were initiated in the slab-adjacent column interfaces. A few flexural cracks were observed in the bottom of the drop panel around the corner column at a load of 7.6 kN (1.71 kip) due to equivalent Vierendeel action. At a load of 10.2 kN (2.30 kip), the first diagonal crack in the slab was formed. It should be pointed out that this diagonal crack was connected with the edges of the drop panels around the longitudinal and transverse adjacent columns as the drop panels have increased the moment capacity of the slab section near to the adjacent column and shifted the most critical
section from the slab-adjacent column interface to the edge of the drop panel. The first yield of the
top reinforcement was observed at a load of 15.4 kN (3.47 kip) and corresponded to a vertical
displacement of 30.0 mm (1.18 in.). When the vertical displacement reached 110.7 mm (4.36 in.),
the first peak capacity $P_{cu}$ was attained at a load of 19.1 kN (4.30 kip) and corresponded to
120.1 % of the design axial load as recommended by DoD$^2$. The major diagonal crack in the slab
became wider with a further increase in the vertical displacement. When the vertical displacement
reached 130.0 mm (5.12 in.), concrete crushing was observed in the top slab local to the corner
column. When the displacement reached 221.3 mm (8.71 in.), which was equivalent to 9.2 % of
the TDR, the load-displacement curve began to ascend again (attributable to tensile membrane
action). Compared with Specimen ND1, no obvious punching failure was observed in the corner
column-slab connection during the test due to the drop panel significantly increasing the effective
depth of the slab. However, similar to ND1, severe flexural cracks were formed in the bottom
surface of the corner drop panel when the vertical displacement reached 350.0 mm (13.8 in.). In
addition, similar to ND1, no cracks were observed in the corner column and joint during the test.

Fig. 9b illustrates the crack pattern development at different performance levels for WD1. The
maximum compressive forces of 8.5 kN (1.91 kip) and 9.0 kN (2.03 kip) were measured in the
transverse and longitudinal directions at the displacements of 90.8 mm (3.57 in.) and 100.7 mm
(3.96 in.), respectively. The maximum horizontal tensile reaction forces of 9.0 kN (2.03 kip) and
7.9 kN (1.78 kip) were measured in the transverse and longitudinal directions at the final stage of
the test, respectively.

ND2—the measured vertical and horizontal reaction force versus the vertical displacement of
the corner joint of ND2 is shown in Fig. 6b. In general, the crack development of ND2 was similar
to that of ND1 and the key points of the test results are listed in Table 2. Thus, only foremost
discrepancies between these two specimens are emphasized here. For ND1, the first diagonal crack
in slab was formed at a load of 6.3 kN (1.42 kip). However, for ND2, the diagonal crack in slab

was formed and passed through the center of the slab at a load of 9.1 kN (2.05 kip). Another
difference between the crack patterns of ND2 and ND1 was that the punching failure occurred at
the corner column-slab connection of ND2 at a displacement of 380.9 mm (15.00 in.) while it
occurred in ND1 at the final stage of the test (410.9 mm or 16.18 in.). In general, the crack pattern
in ND2 was much finer than that in ND1. In ND1, only several discrete diagonal cracks were
formed. However, numerous cracks were observed in between the diagonal cracks in ND2. A
higher slab reinforcement ratio provided in the slab significantly increased the first yield and first
peak capacity of the specimen. When the vertical displacement reached 80.7 mm (3.18 in.), which
was equivalent to 3.36 % TDR, the specimen reached the first peak capacity of 14.3 kN (3.22 kip)
and corresponded to 89.9 % of the design axial load as recommended by DoD². When the vertical
displacement reached 131.3 mm (5.17 in.), tensile membrane action was observed in the load-
displacement curve. The failure mode of ND2 is depicted in Fig. 7 while the crack pattern
development of ND2 was illustrated in Fig. 9c.

**WD2**—the measured vertical and horizontal reaction force versus the vertical displacement of
the corner joint of WD2 is shown in Fig. 6b. In general, the crack development of WD2 was
similar to that of WD1 and the key points of the test results are listed in Table 2. Thus, only
foremost discrepancies between these two specimens are emphasized. For WD1, the first diagonal
crack in slab was formed at a load of 10.2 kN (2.30 kip). However, for WD2, the diagonal crack
was first formed and connected with the edges of the adjacent columns at a load of 14.3 kN (3.22
kip). Similar to ND2, the crack patterns observed in WD2 was much finer than those in WD1.
When the vertical displacement reached 119.9 mm (4.72 in.), which was equivalent to 5.0 % of
TDR, the specimen reached the first peak capacity of 26.8 kN (6.03 kip) and corresponded to
168.6 % of the design axial load as recommended by DoD². When the vertical displacement
reached 200.0 mm (7.87 in.), corresponding to 8.3 % of the TDR, tensile membrane action was
observed in the load-displacement curve. Fig. 8 depicts the failure mode of WD2 while Fig. 9d illustrates the crack pattern development corresponding to different performance levels of WD2.

**ND3**—the measured vertical and horizontal reaction forces versus the vertical displacement of the corner joint of ND3 is shown in Fig. 6c. In general, the crack development of ND3 was similar to that of ND1 and the key points of the test results are listed in Table 2. Thus, only foremost discrepancies between these two specimens were emphasized. For ND1, the first diagonal crack in slab was formed at a load of 6.3 kN (1.42 kip). However, for ND3, the diagonal crack was formed and passed through the center of the slab at a load of 13.4 kN (3.02 kip). Another difference between the cracks patterns of ND3 compared to those of ND1 was that the punching failure occurring in the corner column-slab connection of ND3 was first observed at a displacement of 50.4 mm (1.98 in.), which was before reaching its first peak capacity. However, although the sign of punching shear failure was observed before reaching the first peak capacity, this punching failure deteriorated slowly and did not prevent further redistribution of the load. This is possibly attributed to the special design—integrity reinforcements were installed in both the top and bottom of the slab (refer to Fig. 5). In general, the crack patterns in ND3 were much finer than those in either ND1 or ND2. When the vertical displacement reached 80.9 mm (3.19 in.), which was equivalent to 3.37 % TDR, the specimen reached the first peak capacity of 22.4 kN (5.04 kip) and corresponded to 140.9 % of the design axial load as recommended by DoD². When the vertical displacement reached 120.8 mm (4.76 in.), tensile membrane action was observed in the load-displacement curve. The failure mode of ND3 was similar to ND2 and hence not presented. The crack pattern development of ND3 is illustrated in Fig. 9e.

**WD3**—the measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of Specimen WD3 is shown in Fig. 6c. The first diagonal crack in the slab was formed at the loads of 10.2 kN (2.30 kip) and 14.3 kN (3.22 kip) for Specimens WD1 and WD2, respectively. However, for WD3, the diagonal crack in slab was firstly formed at a load of 17.1 kN...
(3.85 kip). When the vertical displacement reached 110.6 mm (4.35 in.), which was equivalent to 4.6 % of TDR, the specimen reached the first peak capacity of 36.2 kN (8.14 kip) and corresponded to 227.7 % of the design axial load as recommended by DoD. When the vertical displacement reached 221.3 mm (8.71 in.), corresponding to 9.2 % of the TDR, tensile membrane action was observed in the load-displacement curve. It should be pointed out that slight cracks were also observed in the corner joint and corner column at the final stage of the test for this specimen. The failure mode of WD3 was similar to that of WD2 and thus not presented. Fig. 9f illustrates the crack pattern development corresponding to different performance levels of WD3.

DISCUSSION OF THE TEST RESULTS AND DROP PANEL EFFECTS

Comparison of performance of ND-series specimens to corresponding WD-series specimens

Load-displacement relationship

Fig. 6 shows the comparison of the load-displacement relationship of the WD-series specimens with the corresponding ND-series specimens. By comparing the first peak capacity of the two specimens, it can be seen that ND1 can only reach 53.4 % of the design axial load as recommended by DoD, while WD1 can reach 120.1 % of the design axial load. The first peak capacity of WD1 was increased by about 124.7 % compared with that of ND1. Based on the test results, 89.9 % and 168.6 % of the design axial load as recommended by DoD could be achieved by ND2 and WD2, respectively. The first peak capacity of WD2 was enhanced by about 87.5 % compared with ND2. However, Specimens ND2 and WD2 could attain the second peak carrying capacities of 18.5 kN (4.16 kip) and 32.5 kN (7.31 kip) respectively at the final stage of the test. For Specimens ND3 and WD3, 140.9 % and 227.7 % of the design axial load as recommended by DoD could be achieved based on the test results. The first peak capacity of WD3 was increased by about 61.6 % compared with ND3. Moreover, Specimens ND3 and WD3 could attain the second peak carrying capacities of 24.8 kN (5.58 kip) and 40.3 kN (9.07 kip) respectively at the final stage of the tests.
In this study, the initial stiffness was defined as the secant stiffness at the first yield strength. The initial stiffness of Specimens ND1, ND2, and ND3 were 0.24 kN/mm (1.35 kip/in.), 0.39 kN/mm (2.25 kip/in.), and 0.61 kN/mm (3.47 kip/in.), respectively. For Specimens WD1, WD2, and WD3, the initial stiffness were 0.51 kN/mm (2.92 kip/in.), 0.63 kN/mm (3.60 kip/in.), and 0.86 kN/mm (4.94 kip/in.), respectively. Thus, the flat slab incorporated with drop panels could increase the initial stiffness by up to 117.4%.

Energy dissipation

The survival of the structures subjected to the scenario of the loss of a column is related to their ability to dissipate the input energy. In this study, the definition of energy dissipation is the area under the load-displacement curve of each specimen. Fig. 10 shows the energy dissipation capacity of the test specimens. The dissipated energies of Specimens ND1 and WD1 at the final stage of the test were 4.1 kN.m (36.3 kip-in) and 7.6 kN.m (67.3 kip-in), respectively. For Specimens ND2 and WD2, the dissipated energies were 6.3 kN.m (55.8 kip-in) and 11.3 kN.m (100.1 kip-in) respectively. However, the dissipated energies were 8.4 kN.m (74.4 kip-in) and 14.7 kN.m (130.2 kip-in) for ND3 and WD3, respectively. Thus, the incorporated drop panels could increase the energy dissipation capacities by 85.4%, 79.4% and 75.0% for WD1, WD2 and WD3, respectively.

Local behavior - rebar strains

Fig. 11 illustrates the relationship of strain in the slab reinforcement versus vertical displacement of ND1. The locations of strain gauges are illustrated in Fig. 5. As shown in Fig. 11a, ST1 and ST2 were in compression during the test. The maximum compressive strains of ST1 and ST2 were -561 με and -298 με, respectively. This confirmed that the direction of the bending moment of the column strip connected with the corner column changed after the removal of the corner column due to the equivalent Vierendeel action. Moreover, it indicated that the extent of the Vierendeel action was not slack during the test due to the corner column and joint being relatively intact.
during the test. However, for beam-column-slab substructures, the Vierendeel action was sluggish
with increasing damage of the corner joint (Kai and Li\textsuperscript{12}). ST4 recorded the tensile strain of 2331
με and it yielded when the vertical displacement reached 30.9 mm (1.22 in.). For ST5 and ST6, the
measured maximum tensile strains were 2143 με and 1799 με, respectively. They were close to the
yield strain although they had not yielded. However, the measured maximum tensile strain of ST7
was 120 με and this proved that the majority of the force initially resisted by the damaged corner
column was transferred to the adjacent columns while negligible force was transferred to the
interior column (a similar conclusion was reached by Kai and Li\textsuperscript{12}).

Fig. 11b depicts the relationship of strain in the slab bottom reinforcement versus the vertical
displacement. The strain in all bottom reinforcements except SB5 and SB6 were in compression
initially but was altered to be in tension with the increase in the vertical displacement. The tensile
strains in SB1, SB2, SB3 and SB4 were significantly increased when the displacement attained
100.2 mm (3.94 in.). A re-ascending branch was observed in the load-displacement curve at this
displacement stage. The measured maximum tensile strains in SB7 and SB8 were 2143 με and
191 με, respectively. This was due to the fact that the major diagonal crack was passing over the
center of the slab for ND-series specimens. It should be emphasized that the strain in the column
longitudinal reinforcement of ND1 was also measured. The maximum tensile strain and
compressive strain measured in the column longitudinal reinforcement were 363 με and -209 με
respectively. This was consistent with the crack pattern observation—no cracks were observed in
the column and joint regions (in the elastic region) during the test for ND1. For WD1, in general,
the trends of the strain curves were similar to those of ND1 and thus were not repeated herein.

Fig. 12 presents the strain of the reinforcement in the drop panel of WD1 versus the vertical
displacement. The locations of these strain gauges were shown in Fig. 4. The measured maximum
tensile stains in SD1, SD2 and SD3 were 2965 με (beyond the yield strain), 1674 με, and 988 με,
respectively due to equivalent Vierendeel action. However, compressive strains were measured in
SD4, SD5, and SD6 and the maximum compressive strains in SD4, SD5, and SD6 were -295 με, -254 με, and -261 με, respectively.

**Discussion of the punching shear strength of the corner column-slab connection**

As observed in the cracking patterns of the test specimens, unpredicted punching shear cracks were formed in the ND-series specimens. As mentioned above, in order to study the behavior of the specimens with large deformation well, the tested flat slabs were designed not to fail by brittle punching shear. Table 3 summarizes the comparison of the measured punching shear resistance form tested specimens with the values predicted by the punching shear formulations of ACI-08\textsuperscript{17}, Eurocode\textsuperscript{18}, CEB-FIP MC90\textsuperscript{19}, and DIN 1045-1\textsuperscript{20}. For ACI-08\textsuperscript{17} and Eurocode\textsuperscript{18}, both assumptions that the perimeter of the critical section was rounded and straight were calculated due to the observed perimeter of the critical section was rounded. The design formulations of above design code for predicting the punching shear capacities could be found in Gardner\textsuperscript{21}. As shown in the table, the punching shear capacities according to ACI-08\textsuperscript{17} are, in general, significantly overestimated neither assuming the perimeter with rounded corners or simplified assuming the perimeter with straight corners. However, predictions of Euro codes, especially for CEB-FIP MC90\textsuperscript{19}, and DIN 1045-1\textsuperscript{20}, are much closer to the measured values than those of ACI-08\textsuperscript{17}. This was mainly due to the fact that the ACI punching shears formulation accounts neither for the role of the reinforcement ratio nor for the size of the member. Guandalini et al.\textsuperscript{22} have concluded that the nominal punching shear strength decreases for decreasing the flexural reinforcement ratios. Moreover, in general, assuming the perimeters of the critical section were rounded could get closer results compared with assuming the perimeters of the critical section were straight.

**Discussion of the dynamic effect**

As the design guidelines are still developing, the dynamic ultimate strength of each specimen was predicted by a simplified analytical model-capacity curve method in this paper. Then the corresponding dynamic force increase factor (DFIF) of each specimen was determined. DFIF is
defined herein as the ratio of static ultimate capacity to the dynamic ultimate capacity (the peak value of the capacity curve) of each specimen. The capacity curve method was proposed by Abruzzo et al.\textsuperscript{23} based on the conservation of energy. After conducting nonlinear pushover analysis, the load-displacement curve of the structure can be obtained where the area under this curve represents the strain energy in the structure. At the moment where the system achieves a balanced condition, this internal energy will be equal to the external work, defined as the product of the constant applied load (column axial force before damage) and the resulting displacement. If the system does not have adequate ductility to dissipate the required energy, the internal and external work will never balance each other and it will result in a collapse. Thus, a capacity curve may be constructed by dividing the accumulated stored energy by its corresponding displacement. However, it should be noted that the dissipated energy due to damping was not considered in this simplified mode. It is mathematically expressed as:

\[
P_{CC}(u_d) = \frac{1}{u_d} \int_0^{u_d} P_{NS}(u)du
\]  

where \(P_{CC}(u)\) and \(P_{NS}(u)\) are the capacity function and the nonlinear static loading estimated at the displacement demand \(u\), respectively.

Fig. 13 presents the capacity curve and load curve of each specimen. As seen from the figure, the load curves were the intersection of the capacity curves at the displacements of 137.8, 59.2, 50.8 and 24.8 mm (5.43, 2.33, 2.00, and 0.98 in.) for Specimens WD1, ND3, WD2 and WD3, respectively. Thus, these four specimens will not collapse as energy balance can be achieved. However, the load curves in ND1 and ND2 were larger than the dynamic ultimate capacity of the corresponding specimens. Thus, both ND1 and ND2 will totally collapse if the corner support were removed suddenly. Taking ND1 as an example, significant tensile membrane action was observed in the load-displacement or pushover curve (as shown in Fig. 13). However, the increased dynamic ultimate capacity due to this tensile membrane action is very limited (as shown in Fig. 13). Thus,
the contribution of the tensile membrane action in resisting the real dynamic progressive collapse event was probably not so reliable. The predicted ultimate capacity and corresponding DFIFs without considering the tensile membrane action was given in Table 2. As shown in the table, the predicted DFIF for the test specimens ranged from 1.13 to 1.23. The much lower dynamic effects on the ultimate capacity than that assumed in GSA$^3$ may be explained by relative ductile performance exhibited by the test specimens.

**CONCLUSIONS**

Based on the experimental study conducted in this research, the following conclusions were reached:

1. Experimental observation also indicated that one of the potential failure modes for the flat plate structures (ND-series specimens) in resisting progressive collapse caused by the loss of a ground corner column was punching failure, which occurred in the corner column-slab connection. The deterioration of the punching failure was mild and the tests could be continued due to the installation of integrity reinforcement at both the top and bottom of the slab. In addition, the drop panels significantly mitigated the likelihood of such a kind of brittle failure mode. No punching shear cracks were observed in the WD-series specimens.

2. As expected, experimental results indicated that the incorporation of drop panels could significantly improve the overall performance in resisting progressive collapse. The first peak carrying capacities of WD1, WD2 and WD3 (with drop panel) were increased by 124.7 %, 87.5 % and 61.6 % respectively as compared to ND1, ND2 and ND3 (without drop panel).

3. The experimental results indicated that flat slab incorporated with drop panel could increase the initial stiffness and energy dissipation capacity by up to 117.4 % and 85.4 %, respectively.

4. The amount of the slab reinforcement significantly affected the performance of the flat slab
structures in resisting progressive collapse. The first peak carrying capacities increased by 68.2 % and 163.5 % in Specimens ND2 and ND3 respectively compared to ND1. For Specimens WD2 and WD3, the first peak carrying capacities increased by 40.3 % and 89.5 % respectively compared to WD1.

5. The re-ascending branch in the load-displacement curves of the test specimens indicated that tensile membrane action was developed in the slab. Moreover, the second peak carrying capacities of all specimens have exceeded their first peak capacity. However, the punching failure will possibly prevent the development of the tensile membrane action.

6. The predicted dynamic effects for the test specimens ranged from 1.13 to 1.23, which were significantly less than those assumed in the design guidelines of 2 due to the test specimens being relatively ductile. However, correlated dynamic tests should be conducted in future to more accurately evaluate their dynamic progressive collapse performance.

REFERENCES


17 ACI Committee 318, “Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (318R-08),” *American Concrete Institute*, Farmington Hills, MI, 2008, 433 pp.


Fig. 1–Deformation shape of prototype flat plate after one of corner column was lost (no scale)

1: Load cell measure applied load
2: Hydraulic Jack with 600.0 mm stroke
3: Steel column
4: Comp/tension load cell measure horizontal constraint load
5: Steel assembly
6: RC substructure
7-1, 7-2 and 7-3: Rigid steel leg 1, 2 and 3 respectively.
8: Steel weight
9: Displacement transducers to measure deflection of the slab
10: LVDT with 300.0 mm travel to measure deflection of beams
11: A displacement transducer to monitor the horizontal movement of the corner joint

Fig. 2–An overview of a typical specimen in position ready for testing (Note: 1mm=0.0393 in.)
Fig. 3–The detailing of steel assembly

Fig. 4–Dimensions, cross-section details and strain gauge locations of the typical WD-series specimens (in mm)

Note: 1mm=0.0393 in.; T=Deform reinforcing bar; R=Plain reinforcing bar
Fig. 5–Slab reinforcement details and strain gauge locations of ND2 and WD2

Note: 1mm=0.0393 in.
Fig. 6–Comparison of the vertical load and horizontal reaction force versus vertical deflection (ND-series specimens and corresponding WD-series specimens)

Fig. 7–Failure mode of ND2 at final

Fig. 8–Failure mode of WD2 at final
Fig. 9–Observed cracking patterns at different performance levels of test specimens

Fig. 10–Comparison of the energy dissipation capacity of the test specimens
Fig. 11–Strain of slab reinforcement versus vertical displacement in ND1
Fig. 12–Strain of rebar in the drop panels of WD1 versus vertical displacement

Fig. 13–Illustrate the capacity curve and load curve of each specimen
### Table 1-Specimen properties

<table>
<thead>
<tr>
<th>Test</th>
<th>Column stub</th>
<th>Slab thickness</th>
<th>Slab top layer rebar</th>
<th>Slab bottom layer rebar</th>
<th>Drop panel thickness</th>
<th>Design axial force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ND1</td>
<td>R6@125</td>
<td>R6@250</td>
<td>0.58</td>
<td>0.46</td>
<td>None</td>
<td>15.9 (5.88)</td>
</tr>
<tr>
<td>ND2</td>
<td>R6@60</td>
<td>R6@250</td>
<td>1.05</td>
<td>0.99</td>
<td>None</td>
<td>15.9 (5.88)</td>
</tr>
<tr>
<td>ND3</td>
<td>R6@35</td>
<td>R6@70</td>
<td>None</td>
<td>None</td>
<td>15.9 (5.88)</td>
<td></td>
</tr>
<tr>
<td>WD1</td>
<td>R6@125</td>
<td>R6@250</td>
<td>0.89</td>
<td>0.70</td>
<td>None</td>
<td>15.9 (5.88)</td>
</tr>
<tr>
<td>WD2</td>
<td>R6@60</td>
<td>R6@250</td>
<td>1.05</td>
<td>0.99</td>
<td>None</td>
<td>15.9 (5.88)</td>
</tr>
<tr>
<td>WD3</td>
<td>R6@35</td>
<td>R6@70</td>
<td>None</td>
<td>None</td>
<td>15.9 (5.88)</td>
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</tbody>
</table>

Note: 1mm=0.0393 in, R6=Plain rebar with diameter of 6 mm.

### Table 2-Test results

<table>
<thead>
<tr>
<th>Test</th>
<th>First diagonal slab crack kN (kip)</th>
<th>First yield load kN (kip)</th>
<th>First peak capacity kN (kip)</th>
<th>First punching failure mm (in.)</th>
<th>Start to develop tensile membrane kN (kip)</th>
<th>MCHR transverse direction kN (kip)</th>
<th>MCHR longitudinal direction kN (kip)</th>
<th>MTHR in transverse direction kN (kip)</th>
<th>MTHR in longitudinal direction kN (kip)</th>
<th>Second peak capacity kN (kip)</th>
<th>Dynamic strength kN (kip)</th>
<th>DFIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>ND1</td>
<td>6.3 (1.42)</td>
<td>7.3 (1.64)</td>
<td>8.5 (1.91)</td>
<td>410.9 (16.18)</td>
<td>120.3 (4.74)</td>
<td>4.5 (1.01)</td>
<td>6.0 (1.35)</td>
<td>6.1 (1.37)</td>
<td>17.3 (3.89)</td>
<td>6.9 (1.55)</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>ND2</td>
<td>9.1 (2.05)</td>
<td>11.6 (2.61)</td>
<td>14.3 (3.22)</td>
<td>380.9 (15.00)</td>
<td>131.3 (5.17)</td>
<td>4.2 (0.95)</td>
<td>5.7 (1.28)</td>
<td>7.3 (1.64)</td>
<td>8.1 (1.82)</td>
<td>18.5 (4.16)</td>
<td>1.18</td>
<td></td>
</tr>
<tr>
<td>ND3</td>
<td>13.4 (3.02)</td>
<td>20.0 (4.50)</td>
<td>22.4 (5.04)</td>
<td>50.4 (1.98)</td>
<td>120.8 (4.76)</td>
<td>6.6 (1.49)</td>
<td>11.0 (2.48)</td>
<td>9.5 (2.14)</td>
<td>24.8 (5.58)</td>
<td>18.6 (4.19)</td>
<td>1.20</td>
<td></td>
</tr>
<tr>
<td>WD1</td>
<td>10.2 (2.30)</td>
<td>15.4 (3.47)</td>
<td>19.1 (4.30)</td>
<td>None</td>
<td>221.3 (8.71)</td>
<td>8.5 (1.91)</td>
<td>9.0 (2.03)</td>
<td>9.0 (1.78)</td>
<td>24.6 (5.54)</td>
<td>24.0 (6.00)</td>
<td>1.19</td>
<td></td>
</tr>
<tr>
<td>WD2</td>
<td>14.3 (3.22)</td>
<td>22.0 (4.95)</td>
<td>26.8 (6.03)</td>
<td>None</td>
<td>200.0 (7.87)</td>
<td>9.5 (2.14)</td>
<td>10.4 (2.34)</td>
<td>9.0 (2.03)</td>
<td>32.5 (7.31)</td>
<td>23.1 (5.20)</td>
<td>1.16</td>
<td></td>
</tr>
<tr>
<td>WD3</td>
<td>17.1 (3.85)</td>
<td>32.2 (7.25)</td>
<td>36.2 (8.15)</td>
<td>None</td>
<td>221.3 (8.71)</td>
<td>10.9 (2.45)</td>
<td>11.9 (2.68)</td>
<td>13.9 (3.13)</td>
<td>12.5 (2.81)</td>
<td>40.3 (9.07)</td>
<td>32.1 (7.22)</td>
<td></td>
</tr>
</tbody>
</table>

Note: MCHR, MTHR= Maximum compressive horizontal reaction force and maximum tensile horizontal reaction force, respectively. DFIF=Dynamic force increase factor, as the ratio of static ultimate capacity to the peak value measured in the capacity curve.

### Table 3-Comparison of the measured punching shear resistance with the design codes

<table>
<thead>
<tr>
<th>Test</th>
<th>$V_{\text{Test}}$ kN (kip)</th>
<th>$V_{\text{ACI}}^R$ kN (kip)</th>
<th>$V_{\text{ACI}}^S$ kN (kip)</th>
<th>$V_{\text{EC2}}^R$ kN (kip)</th>
<th>$V_{\text{EC2}}^S$ kN (kip)</th>
<th>$V_{\text{CEB}}^S$ kN (kip)</th>
<th>$V_{\text{DFIF}}^S$ kN (kip)</th>
<th>$V_{\text{Test}}$ $V_{\text{ACI}}^S$ kN (kip)</th>
<th>$V_{\text{Test}}$ $V_{\text{EC2}}^S$ kN (kip)</th>
<th>$V_{\text{Test}}$ $V_{\text{CEB}}^S$ kN (kip)</th>
<th>$V_{\text{Test}}$ $V_{\text{DFIF}}^S$ kN (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ND1</td>
<td>17.3 (3.89)</td>
<td>31.9 (7.18)</td>
<td>40.6 (9.14)</td>
<td>16.7 (3.77)</td>
<td>21.3 (4.80)</td>
<td>14.2 (3.20)</td>
<td>15.0 (3.42)</td>
<td>0.54</td>
<td>0.43</td>
<td>1.04</td>
<td>0.81</td>
</tr>
<tr>
<td>ND2</td>
<td>17.2 (3.87)</td>
<td>31.9 (7.18)</td>
<td>40.6 (9.14)</td>
<td>21.1 (4.75)</td>
<td>26.9 (6.05)</td>
<td>17.9 (4.04)</td>
<td>18.9 (4.30)</td>
<td>0.54</td>
<td>0.43</td>
<td>1.04</td>
<td>0.82</td>
</tr>
<tr>
<td>ND3</td>
<td>21.1 (4.72)</td>
<td>31.9 (7.18)</td>
<td>40.6 (9.14)</td>
<td>25.6 (5.76)</td>
<td>32.6 (7.32)</td>
<td>21.7 (4.89)</td>
<td>23.0 (5.23)</td>
<td>0.66</td>
<td>0.52</td>
<td>0.82</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Average: 0.58 0.46 0.89 0.70 1.05 0.99

Note: $V_{\text{ACI}}^R$, $V_{\text{ACI}}^S$, $V_{\text{EC2}}^R$, $V_{\text{EC2}}^S$, $V_{\text{CEB}}^S$, $V_{\text{DFIF}}^S$ = Punching shear strength according to ACI-0817 code by considering the control perimeters with rounded corners and with straight corners, respectively.

$V_{\text{EC2}}^R$, $V_{\text{EC2}}^S$, $V_{\text{CEB}}^S$, $V_{\text{DFIF}}^S$ = Punching shear strength according to EC218 code by considering the control perimeters with rounded corners and with straight corners, respectively.

$V_{\text{DFIF}}^S$ = Punching shear strength according to CEB-FIP99 code and DIN-104599 and by considering the control perimeters with rounded corners and with straight corners, respectively.