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<td>Qian, Kai; Li, Bing</td>
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Resilience of Flat Slab Structures in Different Phases of Progressive Collapse
by Kai Qian and Bing Li

With a spate of high-profile structural collapses leading to severe casualties and economic loss in recent history, there has been a growing interest in understanding the behavior of building structures in resisting progressive collapse. While many experimental and analytical studies have been conducted, they have mainly been focused on beam-column or beam-column-slab frames. This study, on the other hand, focuses on the behavior of reinforced concrete (RC) flat slab structures, as they are also highly vulnerable to collapse. This is especially so given that: 1) flat slab structures may have insufficient stiffness to redistribute the loads initially resisted by the lost column; and 2) the residual load-resisting capacity of remaining structures may not be able to sustain existing service loads after removal of several columns. Two series of multi-panel RC flat slab substructures were tested with two different loading rigs to study the load redistribution behavior and residual resisting capacities of flat slab structures when two different phases of collapse are concerned. The main test results such as load-displacement response, crack pattern, failure modes, and local strain gauge readings were presented and discussed. Based on test results, a series of further analyses were carried out to elucidate the effects of each design parameter.

Keywords: flat slab; load redistribution; progressive collapse; residual load resistance.

INTRODUCTION
The performance of building structures under extreme loading (such as blast environments) has become a major concern in metropolises, especially with the recent surge in terrorist activities targeting public structures. A terrorist attack may result in the failure of a small portion of a building. This initial local failure causes the loss of load-carrying capacity of the building and may lead to the eventual collapse of the entire building or a disproportionately large part of it. This type of collapse is defined as “progressive collapse.” To evaluate the safety of reinforced concrete (RC) structures after the removal of a single column or part of bearing wall, researchers had conducted a number of numerical or experimental studies,\textsuperscript{1-14} in the past decade. After the removal of a single vertical load-bearing element, there are two phases of behavior that should be investigated: the load redistribution capacity in Phase 1 and residual load-resisting capacity in Phase 2. As indicated by Sasani et al.\textsuperscript{4} and Sasani,\textsuperscript{5} when one of the ground columns was removed suddenly, a considerable unbalanced axial force at the joint just above the removed column needs to be redistributed to adjacent vertical load-bearing elements. To evaluate the load redistribution capacity of RC frames following the removal of a single column, a series of experimental tests had been conducted in the form of quasi-static push-down tests\textsuperscript{6,7,10} or simulated dynamic tests.\textsuperscript{9,12,15} Beyond Phase 1 (sustained for approximately 20 milliseconds), the axial force of the upper column above the lost column vanished, assuming the beams and slab in each floor were of similar dimension and reinforcement details. At same time, several researchers\textsuperscript{11,13,16} studied the residual load-resisting capacity (behavior in Phase 2) of the remaining structures with the aid of a multi-point loading system. Existing studies indicated that under concentrated push-down loading regime and multi-point loading regime, for beam-column frames, one could obtain a similar crack pattern and failure mechanism. Thus, a push-down loading regime is used to evaluate the behavior of beam-column frames in Phase 1, and assess their behavior in Phase 2. However, it should be noted that the failure modes and load-resisting mechanisms may be very different for flat slab structures in different phases, as local punching failure may occur at slab-column connections, which is not critical in beam-column frames. Thus, to evaluate the resilience of flat slab structures against progressive collapse, the behavior in different phases of collapse should be assessed individually. For this purpose, two series of tests with two different loading rigs were conducted in this study. The results of P2-series of specimens had been presented in Qian and Li.\textsuperscript{17} This paper can be seen as a continuation of the authors’ previous paper\textsuperscript{17} and it will focus on presentation of the test results of P1-series specimens and discussion of the behavior of flat slabs in different phases.

RESEARCH SIGNIFICANCE
The primary objective of this paper is to experimentally evaluate the load redistribution capacity (LRC) and residual load-resisting capacity (RLRC) of flat slab structures to mitigate progressive collapse triggered by the loss of an interior column. The test results may help researchers and engineers to understand the most critical failure modes of flat slab structures in different phases. This understanding may help the structural engineers to better design new RC flat slab structures or more efficiently retrofit existing flat slab structures to mitigate progressive collapse.

EXPERIMENTAL PROGRAM
Design of test specimens
The controlled prototype slab is gravity-designed with a thickness of 280 mm (11 in.). The average live load is 3.0 kPa (0.4 psi) and the additional dead load is 1.0 kPa (0.15 psi).
No seismic load was considered in the design. The slab flexural reinforcement is designed according to the direct design method (ACI 318-1118). The slab has a flexural reinforcement ratio of 0.25% in the x- and y-directions. According to ACI 318-11, top tensile reinforcement is cut at 0.22 ln from the face of support, where ln is the clear span length. Two integrity reinforcing bars with a diameter of 25 mm (1.0 in.) were installed in both normal directions, which was determined according to Eq. (1) and satisfying the requirement of ACI 352.1R-11.

\[
A_{int} = \frac{0.5w_u l_y}{\phi f_y} \quad (1)
\]

where \(A_{int}\) is the minimum area of the integrity reinforcement in normal directions passing through the column; \(w_u\) is the factored uniformly distributed load; \(f_y\) is the yield strength of the integrity reinforcements; \(\phi = 0.9\); and \(l_x\) and \(l_y\) are the column spacing in the x- and y-directions.

### Design variables in test specimens

The controlled prototype slab corresponds to Specimen P1-70-1.0, which means the specimen belongs to Series P1. It has slab thickness of 70 mm (2.8 in.) and span aspect ratio of 1.0. For specimen designations of remaining specimens, please refer to Table 1. As multi-panel slabs are required, the prototype slab has to be scaled down due to limits of the test facilities in laboratory. The dimensions of the members in prototype and model structure are tabulated in Table 2, whereas the reinforcement ratios are kept the same but within a practical range.

#### Table 1—Test specimen designation and properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Panel dimension (aspect ratio)</th>
<th>Slab top reinforcement</th>
<th>Slab bottom reinforcement</th>
<th>Slab thickness</th>
<th>Drop panel reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1-100-1.0</td>
<td>1500 x 1500 (a = 1.0)</td>
<td>R6@130 ((\rho_{tx,y} = 0.24%))</td>
<td>R6@130 ((\rho_{tx,y} = 0.24%))</td>
<td>100</td>
<td>R6@80 ((\rho_{tx,y} = 1.4%))</td>
</tr>
<tr>
<td>P1-100-1.4</td>
<td>1500 x 2100 (a = 1.4)</td>
<td>R6@130 ((\rho_{tx,y} = 0.24%))</td>
<td>R6@130 ((\rho_{tx,y} = 0.24%))</td>
<td>100</td>
<td>R6@80 ((\rho_{tx,y} = 1.4%))</td>
</tr>
<tr>
<td>P1-70-1.0</td>
<td>1500 x 1500 (a = 1.0)</td>
<td>R6@190 ((\rho_{tx,y} = 0.25%))</td>
<td>R6@190 ((\rho_{tx,y} = 0.25%))</td>
<td>70</td>
<td>R6@80 ((\rho_{tx,y} = 1.4%))</td>
</tr>
<tr>
<td>P1-55-1.0</td>
<td>1500 x 1500 (a = 1.0)</td>
<td>R6@250 ((\rho_{tx,y} = 0.25%))</td>
<td>R6@250 ((\rho_{tx,y} = 0.25%))</td>
<td>55</td>
<td>R6@80 ((\rho_{tx,y} = 1.4%))</td>
</tr>
<tr>
<td>P2-55-1.0</td>
<td>1500 x 1500 (a = 1.0)</td>
<td>R6@250 ((\rho_{tx,y} = 0.25%))</td>
<td>R6@250 ((\rho_{tx,y} = 0.25%))</td>
<td>55</td>
<td>R6@80 ((\rho_{tx,y} = 1.4%))</td>
</tr>
<tr>
<td>P2-P-55-1.0</td>
<td>1500 x 1500 (a = 1.0)</td>
<td>R6@250 ((\rho_{tx,y} = 0.25%))</td>
<td>R6@250 ((\rho_{tx,y} = 0.25%))</td>
<td>55</td>
<td>NA</td>
</tr>
</tbody>
</table>

Notes: Units in mm; 1 mm = 0.0394 in.; NA is not available.

#### Table 2—Dimensions of structural components of P1-70-1.0

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Prototype building</th>
<th>Test model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab thickness</td>
<td>280</td>
<td>70</td>
</tr>
<tr>
<td>Column section</td>
<td>600 x 600</td>
<td>200 x 200 (enlarged for fixed boundary assumption)</td>
</tr>
<tr>
<td>Column spacing</td>
<td>6000/6000 (x-/y-directions)</td>
<td>1500/1500 (x-/y-directions)</td>
</tr>
<tr>
<td>Concrete cover for slabs</td>
<td>25</td>
<td>7</td>
</tr>
<tr>
<td>Concrete cover of drop panel</td>
<td>25</td>
<td>7</td>
</tr>
</tbody>
</table>

Notes: Units in mm; 1 mm = 0.0394 in.

In this study, six one-fourth-scaled specimens were categorized into two series (P1 and P2) to investigate the performance of reinforced concrete (RC) flat slab structures in different collapse phases. Thus, one of the most important variables is the loading regime. Four specimens in the P1-series are tested by concentrated push-down loading regime while remaining two specimens in the P2-series are tested under simulated uniform distributed pressure (UDP) with the help of a 12-point loading system. In the P1-series, the variables of span aspect ratio and slab thickness were investigated. Figure 1 illustrates the details of P1-100-1.4. As shown in the figure, P1-100-1.4 was supported by eight columns, C1 to C8, along perimeter edges of the affected area. The center column, C9, was notionally lost before applying loads. The extension part of the slab in P1-100-1.4 is 375 mm (14.8 in.), \(l/4\), and 525 mm (20.7 in.), \(l/4\) in x- and y-directions, respectively, to simulate surrounding bays.
adjacent to the test structures. As tabulated in Table 1, P1-100-1.0 has a similar thickness and slab reinforcement ratio as P1-100-1.4, but it has a span aspect ratio of 1.0 ($l_x/l_y = 1.0$). Moreover, the only difference among P1-70-1.0, P1-100-1.0, and P1-55-1.0 is the slab thickness. In the P2-series, the variable of drop panel was investigated by comparison of the behavior of Specimens P2-55-1.0 and P2-P-55-1.0. As shown in Table 1, Specimens P1-100-1.4, P1-100-1.0, P1-70-1.0, P1-55-1.0, and P2-55-1.0 have drop panels 35 mm (1.4 in.) thick, constructed above the supporting columns. The single layer reinforcement with a ratio of 1.4% was installed in the drop panel. However, there are no drop panels in Specimen P2-P-55-1.0. The material properties are listed in Table 3.

**Design of test setup**

Two different test setups were designed for P1- and P2-series. A schematic of the test setup of P1-series is shown in Fig. 2. Eight strong steel supports are used to support the perimeter edge of the specimens. However, no support is installed below the center column, C9, as it is notionally removed. The existing axial load in the center column before it was lost is simulated by applying downward displacements at the center column stub through a hydraulic jack with 600 mm (23.6 in.) stroke. To ensure symmetric failure mode and to simulate the horizontal constraints from the upper floors in buildings, a special steel assembly (Item 4 in Fig. 2) is designed. The steel column (Item 3 in Fig. 2) only can freely move in the vertical direction, but the horizontal and rotational freedoms are restrained. For P2-series which was designed to investigate the residual load-resisting capacity (RLRC) of remaining structures after the removal of the column, a 12-point loading system (Item 6 in Fig. 3) is designed to equivalently simulate UDP applied on the slab.

*Fig. 2—Overview of P1-series specimen in position ready for testing. (Note: 1 mm = 0.0394 in.)*

*Fig. 3—Overview of P2-series specimen in position ready for testing.*

In evaluating the RLRC of the structures, it is assumed that the gravity load uniformly distributed over the floor area represents the primary loading pattern.
Instrumentation
Extensive measurement devices were installed both internally and externally to monitor the response of test specimens. A load cell (Item 1 in Fig. 2) was used to measure the applied force. A series of line transducers were placed at various locations below the slab to measure the vertical displacement distribution and deflection shape. Three tension/compression load cells (Item 6 in Fig. 2 and Item 8 in Fig. 3) were used to monitor the load redistribution behavior of the specimens within different phases. For P1-series, tension/compression load cells were installed in three of the steel supports (C1, C2, and C8 in Fig. 1) which are located at one and the same panel. Figure 2 illustrates the installation of tension/compression load cells (Item 6 in Fig. 2). However, for P2-series, the load cells were installed in three of the steel supports (C1, C3, and C7 in Fig. 1), which were located at the corners. A series of strain gauges were mounted on the slab reinforcements to trace the development of different load-resisting mechanisms during tests.

EXPERIMENTAL RESULTS

Global behavior and failure modes
Figure 4(a) shows load-displacement curves of P1-series of specimens. As shown in the figure, P1-100-1.0 attained the maximum load of 93.1 kN (20.9 kip) at a vertical displacement of 10.2 mm (0.4 in.). After achieving the maximum load, the load-resisting capacity suddenly drops to 22.0 kN (4.9 kip), which is approximately 21.5% of the first peak load (FPL). The remaining load resistance stayed almost constant until failure. The failure mode of P1-100-1.0 is shown in Fig. 5. Most of the deformation was concentrated in the interior slab-column connection owing to punching shear failure. No obvious flexural deformation was observed in the slab, although thin flexural cracks were formed at the slab. The strain gauge readings from the slab reinforcements indicated that no yielding was achieved in the slab reinforcements during the test. As the punching shear failure was caused by the top column stub penetrated into the slab, the drop panel has very little effect on the punching shear resistance of the connection, which is very different to the failure modes of normal flat slabs subjected to gravity or cyclic loads. It should be noted that no damage was observed in the surrounding slab-column connections.

P1-100-1.4, similar to P1-100-1.0, exhibited an essentially linear load-displacement response well until the occurrence of punching shear failure in the interior slab-column connection. Different with P1-100-1.0, more elliptic cracks were initially formed at the top surface of the slab. Above 88.9 kN (20.0 kip), punching failure began to occur and the load-resisting capacity began to drop suddenly. Note that the displacement corresponding to the maximum load capacity is 13.4 mm (0.53 in.), which is slightly larger than that of P1-100-1.0 (10.2 mm (0.4 in.)) because of larger column spacing in longitudinal direction of P1-100-1.4. The remaining load resistance of the specimen is measured to 29.4 kN (6.6 kip) and only 33.1% of its FPL. The failure
mode of P1-100-1.4 is shown in Fig. 6. In general, it is very similar to that of P1-100-1.0. As measured, the perimeter of the critical section is rectangular and a distance of 150 mm (5.9 in.), $1.74d$, where $d$ is the effective depth of the slab from the column face. Thus, the perimeter of the punching zone in longitudinal direction is even within the drop panel.

During loading of P1-70-1.0, first circular flexural cracking at the top surface of the slab was observed at a load of 17.9 kN (4.0 kip) and was accompanied by a decrease in load-displacement stiffness. Unlike P1-100-1.0 and P1-100-1.4, yielding of the bottom slab reinforcements was detected in P1-70-1.0 at a load of 43.2 kN (9.7 kip), resulting in a further decrease in the stiffness of the load-displacement curve. After achieving the yield load, it was accompanied by a relatively ductile behavior. The displacement ductility ratio was measured to 4.0. Due to strain hardening of slab reinforcements and compressive membrane action, the load resistance kept increasing until, at a vertical displacement of 52.8 mm (2.1 in.), punching shear failure occurred in the interior slab-column connection. It should be noted that the punching shear failure occurred after yielding of slab reinforcements. Thus, it is more accurate to call it “secondary punching shear failure” to distinguish it from normal punching shear failure, as occurred in P1-100-1.0 and P1-100-1.4. The failure mode of P1-70-1.0 is shown in Fig. 7. Comparing to P1-100-1.0 and P1-100-1.4, wider flexural cracks were observed in P1-70-1.0. Although the flexural deformation of P1-70-1.0 is still not obvious compared to its large column spacing, its flexural deformation is much larger than the aforementioned two specimens. The control
perimeter of critical section of P1-70-1.0 is a distance of 
102 mm (4.0 in.), 1.78d from the column face.

For P1-55-1.0, similar to P1-70-1.0, flexural failure  
was initially dominated its failure mode. However, much  
lower initial stiffness and larger deformation capacity were  
measured in the load-displacement curve. The measured  
yield load is 28.1 kN (6.3 kip) at a displacement 15.6 mm  
(0.6 in.). The failure mode of P1-55-1.0 is illustrated in  
Fig. 8. It can be seen that perceptible flexural deformation  
was observed in this specimen at failure. A series of circular  
flexural cracks were formed at the top surface of the slab  
and severe diagonal flexural cracks occurred in the bottom  
surface. The maximum crack width measured in the top and  
bottom slab exceeded 1.5 and 1.1 mm (0.06 and 0.04 in.),  
respectively. Although severe secondary punching shear  
failure also occurred at a distance of 89.0 mm (3.5 in.) from  
the column face, the perimeter of the punching zone is much  
smaller compared to the aforementioned specimens.

For P2-series specimens, UDP was applied on the top  
surface of the slabs. Thus, in this series of tests, the behavior  
is described in accordance with pressure, rather than loads.  
Figure 4(b) presents the pressure-displacement response of  
P2-series specimens. It should be noted that P2-55-1.0 has  
identical dimensions and reinforcement details as P1-55-1.0.  
During the loads of P2-55-1.0, the first evidence of flexural  
crack, which diagonally connected the interior column and  
corner column, is detected at the bottom surface of the slab  
at a pressure of 2.0 kPa (0.3 psi). Moreover, circular cracks  
were formed at the top surface of the slab at this time. Note  
that the outmost circular cracks with widest crack width did  
not connect the interfaces of surrounding columns, where  
maximum bending moments were predicted, but connect the  
edges of the drop panels of surrounding columns. Yielding  
of the bottom reinforcements near the interior column was  
detected at a pressure of 18.7 kPa (2.7 psi). Further increasing  
applied pressure leads to yielding of the top reinforcement  
near the surrounding columns. The first peak pressure (FPP)  
of 24.9 kPa (3.6 psi) was attained at a displacement of  
80.2 mm (0.3 in.), accompanied by severe concrete crushing  
at the slab-drop panel interfaces. Considerable load resistance re-ascension, accompanied by much wider flexural  
cracks, was observed in the large displacement stage. The  
measured ultimate pressure capacity of P2-55-1.0 at the large  
displacement stage is 26.0 kPa (3.8 psi), which is approximately  
104.4% of its FPP. The failure mode of P2-55-1.0 is shown in Fig. 9. In general, the crack pattern of P2-55-1.0 is  
similar to P1-55-1.0. However, only severe flexural cracks  
were detected at the bottom surface of the slab near to the  
interior column and no punching shear failure was observed  
in the interior slab-column connection.

P2-P-55-1.0 has similar dimensions and reinforcement  
details as P2-55-1.0, but no drop panels were constructed  
above the columns. Much lower stiffness and yield pressure  
capacity (13.0 kPa [1.9 psi]) was measured in P2-P-55-1.0.  
Above the pressure of 18.2 kPa (2.6 psi), punching shear  
failure began to develop in the surrounding slab-column  
connections, resulting in a perceptible decrease in the load  
resistance. Load resistance re-ascension was also detected  
in this specimen after the vertical displacement exceeded to  
80.4 mm (3.2 in.), 1.9d. Figure 10 presents the failure modes  
of P2-P-55-1.0 at the final of test. The crack pattern in the  
top surface is very similar to that of P2-55-1.0, except that  

Fig. 9—Failure mode of Specimen P2-55-1.0: (a) top view; and (b) bottom view.

Fig. 10—Failure mode of Specimen P2-P-55-1.0: (a) top view; and (b) bottom view.
the widest cracks are formed at the interface of surrounding columns, rather than the edges of drop panels. Moreover, severe secondary punching shear failure was detected in all of the surrounding slab-column connections.

Load redistribution behavior

Although the load redistribution behavior of RC planar frames is studied by several researchers, limited studies focused on three-dimensional (3-D) specimens. Figure 11 presents the load redistribution response of test specimens in critical stages. For P1-100-1.0 at the FPL stage, approximately 19% of the axial load, which was initially resisted by the lost center column C9, will transfer into adjacent columns C2 and C8; only 6.4% of the load will transfer to the corner column C1 at this stage. After punching failure occurred, more force was transferred into adjacent columns but fewer loads were transferred into the corner column, perhaps due to the fact that the post-punching resistance was mainly provided by the local catenary action of integrity reinforcements, which go through the reinforcement cages of adjacent columns, as shown in Fig. 12(a). In general, similar behavior

Fig. 11—Load redistribution response of P1-series of specimens in critical stages: (a) P1-100-1.0; (b) P1-100-1.4; (c) P1-70-1.0; (d) P1-55-1.0; (e) P2-55-1.0; and (f) P2-P-55-1.0.

Fig. 12—Load-resisting mechanisms to redistribute loads: (a) local catenary action after punching shear failure; and (b) compressive arch/membrane action.
was observed in P1-100-1.4. However, unlike P1-100-1.0, the force transferred into adjacent columns C2 and C8 are unequal due to different column spacing and stiffness in longitudinal and transverse directions.

As yielding strain was detected in the slab reinforcements of P1-70-1.0 and P1-55-1.0, the behavior of force transference during yield load was also included in the figure. As shown in Fig. 11(c) and 11(d), more force was transferred into adjacent columns while less force transferred into corner column when the load increased from yield load to FPL. This is because the compressive membrane action in P1-70-1.0 and P1-55-1.0 is similar to double compressive arch actions in both longitudinal and transverse directions, as shown in Fig. 12(b).

As the tension/compression load cells were installed in the corner supports C1, C5, and C7 in P2-series of specimens, only the load redistribution response of corner columns was monitored. Unlike P1-70-1.0 and P1-55-1.0, more force was transferred into corner columns when the load increased from yield load to FPP. This is because a larger area was involved to develop compressive membrane action in the P2-series of specimens where UDP was applied. Moreover, the force transferred into corner columns was increased when the load achieved the stage of load-re-ascension. This is due to more severe damage occurring in the slab close-in at adjacent columns at this stage—that is, severe punching shear failure occurred in the adjacent slab-column connection for P2-P-55-1.0.

Basic analysis indicated that the increase of axial force in adjacent columns may not exceed 20% and 30% in Phases 1 and 2, respectively. The axial force is less than $0.4f'_cA_g$ for RC columns in normal design, where $f'_c$ is compressive strength of concrete, and $A_g$ is the gross area of the column section. Thus, the compressive failure of adjacent columns due to increasing axial force after force redistribution is not as critical in RC buildings. However, for steel structures, more attention is needed.

Strain gauge results

Figure 13 presents the strain distribution in the slab reinforcements of test specimens at the FPL stage. It can be seen that no yielding was detected in slab reinforcements of P1-100-1.0 at the FPL. Similar results were observed in P1-100-1.4. The failure mode of these two specimens agrees well (failure was dominated by punching shear). However, severe yielding was recorded in both top and bottom slab reinforcements of P1-55-1.0, as shown in Fig. 13(b). Compared to P1-55-1.0, more reinforcement achieved yielding strain in P2-55-1.0 due to simulated multi-point loading regime, which led to finer cracks and more uniform deformation. As shown in Fig. 13(d), yielding was achieved in slab reinforcements of P2-55-1.0, although severe punching shear failure was observed in adjacent slab-column connections. If a thicker slab is designed in P2-55-1.0, sudden, brittle punching shear failure may occur before reinforcing bar yielding. Thus, punching shear failure occurring in adjacent slab-column connections after force redistribution also requires more attention. As the damage of slab reinforcement is very difficult to monitor completely based on the limited number of pre-installed strain gauges, the Frequency...
Modulated Thermal Wave Imaging (FMTWI) technique, which was proposed by Mulaveesala et al.\textsuperscript{20} could be used as an alternative method.

**ANALYSIS AND DISCUSSION**

**Yield line prediction**

Yield line prediction is used to predict the flexural strength of test specimens. Figure 14(a) presents a typical yield line configuration of the specimens with drop panels based on the observed crack patterns. Similar to Qian and Li,\textsuperscript{17} for square specimens with an aspect ratio of 1.0, the internal virtual work due to rotations of the yield lines is

\[
W_I = \sum (m_s + m_h) + \sum \delta + \sum \delta^2 \sin \theta
\]

(2)

where \(W_I\) is the internal work due to strain energy; \(\delta\) is virtual displacement at the center column; \(h_1\) and \(h_2\) are the length of the centerline of Segments A and B, respectively; \(\gamma_1 = 2h_1\tan(\alpha/2)\) and \(\gamma_2 = 2h_2\tan(\beta/2)\) are the length of hogging yield line in Segments A and B, respectively; the measured angle of \(\alpha\) and \(\beta\) range from 65 to 73 degrees and 17 to 25 degrees, respectively. For simplicity, the values of \(\alpha\) and \(\beta\) are assumed to be 69 and 21 degrees, respectively; and \(m_s\) and \(m_h\) are, respectively, the yield moment of the sagging and hogging yield line, which could be determined by Eq. (3)\textsuperscript{21}

\[
m_s = A_s f_y \left( d_s - 0.59 A_s \frac{f_y}{f_{yc}} \right)
\]

(3)

where \(A_s\) is the tensile slab reinforcement per unit width; and \(d_s\) is the slab effective depth.

For P1-100-1.4, as shown in Fig. 14(b), the internal virtual work due to rotations of the yield lines is

\[
W_I = 4\delta \left[ m_s \left( \frac{\gamma_1}{h_1} + \frac{\gamma_2}{h_2} \right) + 2m_h \left( \frac{h_1}{h_2} \sin \frac{\theta}{2} + \sin \frac{\beta}{2} \right) \right]
\]

(4)

where \(h_3\) is the length of the centerline of Segment C; \(\gamma_3 = 2h_3\tan(\theta/2)\) is the length of hogging yield line of Segment C; and the measured angles of \(\alpha\), \(\beta\), and \(\theta\) are 46, 16, and 102 degrees, respectively, in P1-100-1.4.

For P1-series specimens with concentrated loads, the external virtual work is

\[
\sum W_E = P\delta
\]

(5)

where \(W_E\) is the external work due to applied external load; and \(P\) is the external concentrate load.

For P2-series specimens, the external virtual work is

\[
\sum W_E = q \left( \frac{2\delta}{r_h} h_1 + \frac{\delta}{r_h} h_2 \right)
\]

(6)

where \(q\) is the external pressure.

The predicted yield load of P1-100-1.0, P1-100-1.4, P1-70-1.0, and P1-55-1.0 are 112.5, 114.3, 50.1, and 28.1 kN (25.3, 25.7, 11.3, and 6.3 kip), respectively. The FPL of P1-100-1.0 and P1-100-1.4 are 93.1 and 88.9 kN (20.9 and 19.9 kip), respectively. Thus, the analytical prediction further confirmed that the behavior of P1-100-1.0 and P1-100-1.4 is controlled by the punching shear failure rather than flexural failure. Based on the design equation proposed by EN1992-1-1:2005\textsuperscript{22} the punching shear resistance of interior slab-column connection of P1-100-1.0 and P1-100-1.4 is 85.7 kN (19.3 kip). Comparing the FPL of these two specimens, the design equation proposed in EN1992-1-1:2005\textsuperscript{22} is reliable.
Residual load resistance of P2-series

As the P2-series specimens were subjected to gravity pressure after removal of a center column, the measured behavior was defined as the residual behavior while the recorded load-resisting capacity is thereafter referred to as the residual load-resisting capacity (RLRC). Figure 15 shows the yield-line configuration of P2-55-1.0 at service load condition and without any column missing. For simplicity, the value of α and β are assumed to be 69 and 21 degrees, respectively. The yield pressure of P2-55-1.0 and P2-P-55-1.0 are 20.9 and 14.1 kPa (3.0 and 2.0 psi), respectively, which agree well with the test results. It should be noted that the measured yield pressure of P2-55-1.0 and P2-P-55-1.0 are 18.7 and 13.0 kPa (2.7 and 1.9 psi), respectively.

Residual load resistance of P2-series

The difference among P1-100-1.0, P1-70-1.0, and P1-55-1.0 is the slab thickness. The measured load-displacement curve indicated that P1-100-1.0 increased the FPL by 85.8% and 150.9% compared to P1-70-1.0 and P1-55-1.0. However, P1-100-1.0 only increased the dynamic load-resisting capacity of P1-70-1.0 and P1-55-1.0 by 45.3% and 99.7%, respectively. This is because of the increase of thickness, which changed the failure mode from ductile flexural failure to brittle punching shear failure and reduced the deformation capacity significantly. As indicated in Tsai8 and Qian and Li,10 the dynamic increase factor is inversely proportional to the deformation capacity of the structures. The dynamic load-resisting capacity of test specimen could be calculated by Eq. (9), as suggested by Izzuddin et al.23

\[
P_{CC}(u) = \frac{1}{u_d} \int_0^{u_d} P_{NS}(u)du
\]  
(9)

where \(P_{CC}(u)\) and \(P_{NS}(u)\) are the capacity function and the nonlinear static loading estimated at the displacement demand \(u\), respectively. In Eq. (9), \(\int_0^{u_d} P_{NS}(u)du\) represents the accumulated area under the nonlinear static load-displacement curve at displacement \(u_d\). Thus, the capacity curve method could be understood as the dividing the accumulated area under the nonlinear static load-displacement curve by its corresponding displacement \(u_d\).

P1-100-1.4 has identical thickness as P1-100-1.0 but has a span aspect ratio of 1.4. No significant difference was observed between these two specimens, as both specimens were controlled by the punching shear failure at the interior slab-column connection.

Effects of different phases of progressive collapse concerned

P1-55-1.0 and P2-55-1.0 had identical dimensions and reinforcement details but different phases of collapse (two different loading protocols were used). Although both specimens failed after the yielding of slab reinforcements, P2-55-1.0 developed finer cracks and a much larger deformation capacity. Moreover, secondary punching shear failure was detected in P1-55-1.0, whereas it was not critical in P2-55-1.0. This is because the drop panel could not enhance the punching shear resistance of interior slab-column connection when the penetrating is from slab top surface to bottom for P1-55-1.0. However, comparing the behavior of P2-P-55-1.0 with P2-55-1.0 confirmed that the drop panels do increase the punching shear resistance of
adjacent slab-column connections significantly and prevent the occurrence of punching shear failure effectively. The test results further confirmed that, unlike moment-resisting frames, a different test setup and loading method should be designed when different phases of collapse are studied in flat slab structures.

CONCLUSIONS

The experimental study conducted in this research has derived the following conclusions:

1. As expected, in Phase 1, a thicker slab could increase the initial stiffness and load resisting capacity of specimens significantly. However, brittle punching shear failure may dominate the failure mechanism of specimens with thicker slabs (P1-100-1.0 and P1-100-1.4), and thus reduce their deformation capacities. Therefore, the improvement of dynamic behavior of the specimens with thicker slabs will not be as significant as their quasi-static behavior.

2. For specimens with thinner slabs (P1-70-1.0 and P1-55-1.0), secondary punching shear failure occurred after slab reinforcements reached yielding strain. Considerable load enhancement was detected in the load-displacement response after yielding of slab reinforcement due to strain hardening and compressive membrane action. All specimens in the P1-series achieved similar remaining load-resisting capacity after punching shear failure, as a similar extent of local catenary action was developed in the integrity reinforcements.

3. The effects of different loading protocols corresponding to different phases of collapse were evaluated by comparison of the behavior of P1-55-1.0, P2-55-1.0, and P2-P-55-1.0. Secondary punching shear failure occurred at the interior slab-column connection of P1-55-1.0, which was tested under concentrated push-down loading regime. The test results indicated that drop panels had limited contribution on the punching shear resistance of the interior slab-column connection of the specimen. However, no obvious punching shear failure was observed in any of the connections of P2-55-1.0, which was subjected to a multi-point push-down loading regime. The failure mode of P2-P-55-1.0, which was also subjected to a multi-point push-down loading regime, indicated that punching shear failure may occur at the adjacent slab-column connection for specimens in Phase 2. The reason no punching shear failure was observed in P2-P-55-1.0 is the drop panels increased the punching shear resistance of adjacent slab-column connections significantly.

4. Analytical results indicated that yield-line method could predict the yield load of tested specimens, which were controlled well by flexural failure. In addition, analytical analysis indicated that P2-55-1.0 and P2-P-55-1.0 could achieve 79.2% and 100.7% of the yield load capacity of the corresponding specimens in normal service condition (no column was removed), respectively. This important finding is due to the missing column increasing the column spacing of the slab and changing the failure mode of the specimens from brittle punching shear failure to ductile flexural failure. More tests should be carried out to further understand this finding.

FUTURE WORKS

As relatively small-scaled multi-panel specimens were carried out to eliminate the size effects on the results of test specimens, more large-scaled multi-panel specimens should be carried out. Moreover, more dynamic tests should be carried out, as sudden column removal may generate dynamic effects, which may aggregate the damage.

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