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Slab Effects on Response of Reinforced Concrete Substructures after Loss of Corner Column

by Kai Qian and Bing Li

In typical cast-in-place construction, beams, columns, and slabs act as a single structural unit. Ignoring the slab contribution to the strength and ductility of beams will result in a significant underestimation of the vertical force resistance. The influence of the slab on the strength of the floor system under imposed vertical deformation is significantly greater than that anticipated by the interpretation of the current provisions for effective slab widths acting as a flange in a T-beam analysis. Therefore, to quantify the contribution of the slab toward progressive collapse of building structures in the blast environment, two series of specimens (F and S) 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, were discussed. Comparison of the performance of these two series of specimens indicated that incorporating the reinforced concrete (RC) slab into the beam-column substructures would increase the ultimate resistance capacity by up to 63.0% and significantly reduce the likelihood of progressive collapse.

Keywords: beam-column; corner; progressive collapse; reinforced concrete; slab; substructures.

INTRODUCTION

The performance of building structures, especially “iconic buildings,” under extreme loading (such as blast environments) has been a major concern in metropolises, especially with the surge in terrorist activities targeting public structures. In many urban settings, the proximity to unregulated traffic brings the terrorist threat to or within the perimeter of the building. In general, a terrorist attack may lead to a failure of a small part of the building. When an initial local failure causes the loss of gravity load capacity of the structural frame, it 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.” While it is not possible for a building to be engineered and constructed to be absolutely risk-free, it is important that structural designers take appropriate precautions and necessary measures to limit the risk of progressive collapse where the damage potential is disproportionate to the cause. To assess the safety of an existing reinforced concrete (RC) structure in terms of the remaining stiffness and load-carrying capacity, engineers need reliable structural models and analytical tools.

Although several design codes and guidelines have formulated their own approach to mitigate progressive collapse of a structure, the criteria in these codes and design guidelines are mainly based on numerical simulation and obviously need to be further corroborated by experimental data. Several in-place tests have been conducted to study the performance of RC structures with precast slabs under the scenario of being subjected to the loss of a column. A number of specimen tests on beam-column subassemblages for progressive collapse have also been conducted in past years. These tests have significantly improved the state-of-the-art quality of protective design and added to the knowledge pool on the progressive collapse behavior of RC structures. However, the majority of the previous specimen tests were carried out on planar specimens without monolithic floor slabs and transverse beams. In typical cast-in-place construction, beams, columns, and slabs are cast simultaneously and therefore act as a single structural unit. Ignoring the effect of the slab on the stiffness and strength of the beams will most likely result in a significant underestimation of the vertical force resistance capacity. The influence of the slab on the strength of the floor system under imposed vertical deformation is significantly greater than that anticipated by the interpretation of the current provisions for effective slab widths acting as a flange in a T-beam analysis. Moreover, few experimental studies have been conducted to study the performance of the RC frame for progressive collapse under the scenario of the loss of a corner column. It is one of the most critical cases due to the special location in the structure and benefits least from the catenary action. Therefore, two series of tests (F and S) were conducted at Nanyang Technological University (NTU), Singapore, to investigate the performance of the RC substructures. By comparing the failure mechanisms and load-displacement response of these two specimen series, the effect of the slabs on the performance of the RC substructures can be determined.

RESEARCH SIGNIFICANCE

The performance of the substructures with and without the RC slab was evaluated in this study. The primary objective of this study is to investigate the slab effect on the vertical load-displacement relationship, crack pattern, and failure mechanism of the substructure by a comparison of the results. This study can help the structural engineer to effectively design new structures and accurately evaluate the ability of existing structures to mitigate progressive collapse.

EXPERIMENTAL PROGRAM

Design of test setup

Figure 1 demonstrates the deformation shape of a nine-story RC frame after one of the ground corner columns was lost. As shown in Fig. 1, the deformation was concentrated in the corner panels. Therefore, one typical critical panel (corner panel in the second story) was extracted and studied. Two different test setups were designed for the S and F specimen series. A schematic of the test setup of the S-series
specimens is shown in Fig. 2. Three rigid steel legs were used to support the slab and each steel leg was connected with a 75.0 mm (3.0 in.) thick strong plate through four $\Phi 27$ bolts. The steel plates were fastened to the strong floor using pretensioned steel rods. Seven steel weight assemblies were applied on the extended part of the slab to simulate the influence of continuity of the surrounding slab on the response of the S-series specimens. 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 mm (23.6 in.) stroke.

As concluded by Sasani, three-dimensional (3-D) Vierendeel action was identified as the major mechanism for the redistribution of loads in the frame under the scenario of the loss of a corner column. Vierendeel action would only develop after deformation of the beam-column frame system. Only one story of the structure, however, was extracted and modeled in this study. Therefore, it was extremely challenging to determine a method of properly applying the Vierendeel action on the substructure to simulate it in a real frame system. Figure 3 illustrates the detailing of the steel assembly used to apply the Vierendeel action approximately on the tested specimens. One strong steel column was connected with the corner stub of the RC specimen by premerged bolts. Four high-strength, high-stiffness steel pins were used to apply prescribed partial rotational and horizontal constraints in each direction. In other words, the steel column could freely move in the vertical direction but the rotational and horizontal freedoms were partially restrained.

To design the steel assembly accurately to provide prescribed partial rotational constraints on the beam end, 3-D finite element models (FEMs) were prepared by using the commercial software ABAQUS. The model was initially validated with the results obtained by Sasani et al. It was subsequently used to design the allowance between the steel pin and the hole, as shown in Fig. 3. The FEM result indicated that the center of the joint just above the lost column had maximum outward horizontal movement of approximately 7.2 mm (0.28 in.), whereas the vertical displacement ($D_{1}$) was approximately 180.0 mm (7.09 in.). The allowance between the steel pin and the hole was designed as follows:

1: Load cell measure applied load
2: Hydraulic Jack with 600 mm stroke
3: Steel column
4: Comp/tension load cell measure horizontal constraint load
5: Steel assembly
6: RC substructure
7-1, 7-1 and 7-3: Rigid steel leg 1, 2 and 3 respectively.
8: Steel weight
9: Line LVDTs to measure deflection of the slab
10: LVDTs with 300 mm travel to measure deflection of beams
11: LVDTs with 25 mm travel to monitor the deformation of steel legs

---

**Fig. 1**—Deformation shape of prototype structures after one of corner columns was lost (no scale).

**Fig. 2**—Overview of slab specimen in position ready for testing. (Note: 1 mm = 0.03937 in.; LVDTs are linear variable displacement transducers.)
\[
\phi = \frac{H_1}{TV} = \frac{H_1}{V + D_1} = \frac{7.2}{625 + 180} = 8.9 \times 10^{-3}
\]

(1)

\[
\delta = \frac{\sqrt{\overline{V}} \times \overline{\phi}}{2} = \frac{350 \times 8.9 \times 10^{-3}}{2} = 1.56
\]

(2)

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 a 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 the beginning of the test; and \( D_1 \) is the vertical displacement.

where \( \overline{V} \) 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 mm (1.6 in.), while the diameter of the hole in the steel box was designed to be 43 mm (1.7 in.), as shown in Fig. 3.

For F-series specimens (as shown in Fig. 4), vertical, axial, and rotational constraints were provided at the adjacent, enlarged columns to simulate the fixed-end boundary condition provided by the surrounding structural elements.

**Experimental substructures**

The nonseismically detailed prototype RC moment-resisting frames were designed according to provisions of Singapore Standard CP 65,\(^{12}\) whereas the seismically detailed prototype frame was designed according to ACI 318-08.\(^{13}\) The dead load of the prototype structure due to the 210.0 mm (8.3 in.) thick slab was 5.1 kPa (0.7 psi). The additional dead load was assumed to be 1.0 kPa (0.2 psi). The live load was assumed to be 2.0 kPa (0.3 psi). One-third-scale substructures were cast and tested in this study. A uniform pressure of 6.6 kPa (1.0 psi) based on the loading combination (DL + 0.25LL) was applied on the surface of the scaled slab. A uniform load of 3.1 kN/m (0.2 kip/ft) was applied on the edge beams of the slab to simulate the line load of the infill wall. The required load resistance for each specimen recommended by the GSA\(^{3}\) guidelines is listed in Table 1.

Figure 5 demonstrates the typical beam and column dimensions and reinforcement details of Specimens S1 and S2, and Fig. 6 illustrates the slab reinforcement details of S1 and S2. The concrete cover of the beam, column, and slab are 10, 20, and 7 mm (0.394, 0.787, and 0.276 in.), respectively. As shown in Fig. 5, for S-series specimens, there is one corner column stub, three adjacent enlarged columns, and four RC beams cast monolithically. The corner column stub repre-
senting the removed column was a 200.0 mm (7.9 in.) square for all specimens while the adjacent, enlarged columns were 250.0 mm (9.8 in.) squares to ensure failure would not occur in these adjacent, enlarged columns. Four F25 bolts were merged in each adjacent, enlarged column, connecting them with the steel legs.

The dimensions and reinforcement details of each specimen are given in Table 1. As shown in the table, S1 and S3 are nonseismically designed and detailed. Thus, 4 T10 was doubly longitudinally reinforced in the beams while transverse reinforcements were in the form of hoop stirrups with

<table>
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<tr>
<th>Test</th>
<th>Column stub</th>
<th>Beam-T</th>
<th>Beam-L</th>
<th>Beam-T</th>
<th>Beam-L</th>
<th>Joint</th>
<th>Beam-T</th>
<th>Beam-L</th>
<th>Required resistance, kN (kips)</th>
<th>Slab reinforcing bar</th>
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<tr>
<td>F1</td>
<td>Height = 450 mm Cross section = 200 x 200 Reinforcement ratio = 2.0%</td>
<td>Type a*</td>
<td>Type a*</td>
<td>0.87%</td>
<td>0.87%</td>
<td>None</td>
<td>0.31%</td>
<td>0.31%</td>
<td>33.7 (7.6)</td>
<td>None</td>
</tr>
<tr>
<td>F2</td>
<td>Type a*</td>
<td>Type a*</td>
<td>1.47%</td>
<td>1.47%</td>
<td>0.49%</td>
<td>0.95%</td>
<td>0.95%</td>
<td>33.7 (7.6)</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>F3</td>
<td>Type a*</td>
<td>Type b*</td>
<td>0.87%</td>
<td>0.75%</td>
<td>None</td>
<td>0.31%</td>
<td>0.36%</td>
<td>40.4 (9.1)</td>
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</tr>
<tr>
<td>S1</td>
<td>Type a*</td>
<td>Type a*</td>
<td>0.87%</td>
<td>0.87%</td>
<td>None</td>
<td>0.31%</td>
<td>0.31%</td>
<td>33.7 (7.6)</td>
<td>0.4%</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>Type a*</td>
<td>Type a*</td>
<td>1.47%</td>
<td>1.47%</td>
<td>0.49%</td>
<td>0.95%</td>
<td>0.95%</td>
<td>33.7 (7.6)</td>
<td>0.4%</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>Type a*</td>
<td>Type b*</td>
<td>0.87%</td>
<td>0.75%</td>
<td>None</td>
<td>0.31%</td>
<td>0.36%</td>
<td>40.4 (9.1)</td>
<td>0.4%</td>
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Notes: 1 mm = 0.0394 in.; Beam-T, Beam-L are transverse beam and longitudinal beam, respectively; Type a* is clear span = 2175 mm cross section = 180 x 100; Type b* is clear span = 2775 mm cross section = 210 x 100.

Fig. 5—Dimensions, cross-section details, and strain gauge locations of Specimens S1 and S2 (in mm). (Note: 1 mm = 0.0394 in.; T is deformed reinforcing bar; R is plain reinforcing bar.)

Fig. 6—Slab reinforcement details and strain gauge locations of Specimens S1 and S2 (in mm). (Note: 1 mm = 0.0394 in.)
90-degree bends. Moreover, no transverse reinforcement was provided in the joint region. However, S2 was seismically designed and detailed. Thus, 4 T13 was doubly longitudinally reinforced in the beams while transverse reinforcements were provided via hoop stirrups with 135-degree bends and transverse reinforcements were provided in the joint region. It should be noted that S1 and S2 have an equal span in the longitudinal and transverse directions, whereas S3 has 2775 and 2175 mm (109.3 and 85.6 in.) clear span in the longitudinal and transverse directions, respectively. Thus, the beam cross section in the longitudinal beam of S3 is 210 x 100 mm (8.27 x 3.94 in.). Specimens F1, F2, and F3 were corresponding to S1, S2, and S3, respectively. As shown in Table 1, similar details were provided in the beams and columns of the F-series specimens as were provided in the corresponding S-series specimens, while no RC slab was incorporated. Moreover, for the F-series specimens, slabs were not incorporated and fixed ends were assumed in the adjacent, enlarged columns. Thus, the longitudinal and transverse interior beams were eliminated in the model (refer to Fig. 5). High-yield-strength steel was used for the longitudinal reinforcement (T16, T13, and T10), whereas mild steel was used for the transverse reinforcement (R10 and R6). The average concrete compressive strength was approximately 32.0 MPa (4640 psi).

**Instrumentation**

Extensive measurement devices were installed both internally and externally to monitor the response of the test specimens. A total of 100 and 130 data channels were active during the testing process for the F- and S-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 box and were used to measure the horizontal reaction of the box in each direction. For the F-series specimens, three compression/tension load cells were installed in both fixed ends. Two load cells were vertical and were used to measure the vertical reaction and moment at the fixed end. The horizontal cell was used to measure the horizontal constraint force of the fixed end. A series of LVDTs and linear potentiometers were also placed at various locations of the substructure to measure the different types of internal deformation, such as fixed-end rotation, curvature, and diagonal deformations. A total of approximately 60 strain gauges were mounted on the reinforcement at strategic locations to monitor strain variation along the beams, corner column, and joint during the test regime. The specific locations of these strain gauges are the same as those of the corresponding S-series specimens, which are shown in Fig. 5. For the S-series specimens, 15 LVDTs were placed vertically to monitor the deflection of the slab and beams. One LVDT was installed horizontally to measure the horizontal movement of the corner column stub with increasing vertical displacement. Strain gauges were not only installed in the beam and column but were also installed in the slab reinforcement. Figure 6 shows the locations of the strain gauges placed in the slabs of the S-series specimens.

**EXPERIMENTAL RESULTS OF BEAM-COLUMN SUBSTRUCTURES**

Two series of specimens (F and S) were constructed and tested to evaluate the slab effect on the performance of the RC frame for progressive collapse caused by the loss of a ground corner column with different design detailing and span aspect ratios. The key points of the test results of the six specimens are summarized in Table 2 and discussed in the following.

**Global behavior**

**F1**—The measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of F1 with different performance levels is shown in Fig. 7(a). Five performance levels at significant parts of the test were identified. PL1, PL2, PL3, PL4, and PL5 represent the first flexural crack, the first yield of the beam longitudinal reinforcement, the ultimate capacity, the normal failure stage (which is defined as the resistant capacity dropped by more than 20% of the ultimate capacity), and when the tensile catenary/membrane action began to develop, respectively. The first crack was observed at the beam end near the fixed end (BENF) at a load of 4.3 kN (1.0 kips). Following the first crack, joint shear cracks were observed at a load of 22.1 kN (5.0 kips) before the first hinges developed in the negative moment reinforcement in the BENF, which occurred at a load of 22.5 kN (5.1 kips). After plastic hinges were formed in the BENFs, the beam bottom longitudinal reinforcing bar in the beam end near the corner joint (BENC) reached maximum tensile strain. Corner joint shear cracks become wider and the ultimate capacity $P_{cu}$ was achieved at a load of 25.8 kN (5.8 kips) and corresponded to 77.6% of the load specified by GSA that is required to resist progressive collapse. The compressive reaction force was measured in the fixed end by horizontal tension/compression load.
The compressive reaction force was ignored before the first crack was observed in the beams of the specimens, but significantly increased after first crack loading. The relationship between the horizontal reaction force and deflection is similar to the relationship between the vertical load and deflection. However, the maximum horizontal compressive load of 19.7 kN (4.4 kips) was reached at a deflection of 54.0 mm (2.1 in.). This is equivalent to 2.5% of the tip displacement ratio (TDR), which was defined as the ratio of vertical displacement of the center of the corner stub to the clear span of beam. The ultimate capacity, however, was reached at a deflection of 44.0 mm (1.7 in.), which was equivalent to 2.0% of TDR. The horizontal compressive reaction force began to fall due to severe shear cracks that had occurred in the corner joint. The severe shear cracks reduced the rotational constraint on the BENC, as indicated by the decrease in the measured vertical load resistance and horizontal compressive reaction force. After the joint shear cracks widened, the strains of the beam longitudinal reinforcing bar in the BENC began to decrease, whereas the strain of the longitudinal reinforcing bar in the BENF increased rapidly. This indicated the change in the resistance mechanism in the substructure to a cantilever beam. With a further increase in the deflection, concrete crushing was observed in the BENF and concrete splitting was observed in the corner joint at the deflection of 160 and 240 mm (6.3 and 9.4 in.), respectively. When the reinforcing bar within the top of the beam near to the fixed end finally fractured, it caused the vertical load resistance to drop to zero. The failure mode of F1 is depicted in Fig. 8.

S1—The measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of S1 is shown in Fig. 7(a). At a load of 18.0 kN (4.1 kips), flexural cracks were initiated in the slab. No cracks were observed within the beam and corner joint regions. At a load of 22.0 kN (5.0 kips), a few flexural cracks were observed in the beam end near the adjacent column (BENAC), and the first diagonal crack in the slab was formed and passed through the center of the slab. At a load of 24.0 kN (5.4 kips), the first diagonal shear cracks occurred in the corner joint. When the applied load was increased further, more diagonal cracks formed in the slab. When the vertical displacement reached 43.0 mm (1.7 in.)—equivalent to 2.0% of TDR—the specimen reached the maximum horizontal compressive load of 21.0 kN (4.7 kips). When the vertical displacement reached 56.0 mm (2.2 in.)—equivalent to 2.6% of TDR—the specimen reached the ultimate capacity of 39.1 kN (8.8 kips) and corresponded to 116.0% of the load specified by GSA3 that is required to resist progressive collapse. With
a further increase in the vertical displacement, the major diagonal crack in the slab and corner joint diagonal cracks became wider and considerable torsion deformation was observed in the edge beams. When the vertical displacement reached 90 mm (3.54 in.), concrete crushing occurring in the BENAC and concrete splitting occurring in the corner joint were observed at a deflection of 120 mm (4.7 in.). When the displacement reached 200 mm (7.9 in.)—that is, equivalent to 9.2% TDR—a diagonal crack in the slab penetrated through the depth of the slab and the load-displacement curve began to ascend again (attributable to tensile membrane action). Figure 9 illustrates the crack pattern development at different performance levels of the slab of S1.

F2—The measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of F2 is shown in Fig. 7(b). In general, the crack development of F2 was similar to that of F1 and the key points of the test results are listed in Table 2. Thus, only the foremost discrepancies between these two specimens are emphasized herein. For F1, the crack width in the beam bottom near the corner joint did not become wider after severe joint cracks had occurred. For F2, however, more cracks developed at the bottom of BENF and these cracks became wider after severe joint shear cracks appeared. Another difference between the crack patterns of F2 and F1 was that the core joint concrete remained relatively intact due to the transverse reinforcement effectively confining the concrete in the joint of F2 after the joint concrete cover split at a deflection of 280.0 mm (11.0 in.). A higher longitudinal reinforcement ratio provided in the beams significantly increased the first yield load and ultimate capacity of the specimen, whereas the higher transverse reinforcement ratio provided in the beam potential plastic hinge zones delayed the concrete crushing and bulking of the beam bottom longitudinal reinforcement in the BENF. When the vertical displacement reached 53.0 mm (2.2 in.)—that is, equivalent to 2.4% TDR—the specimen reached the ultimate capacity of 36.5 kN (8.2 kips) and corresponded to 108.0% of the load specified by GSA 3 that is required to resist progressive collapse. When the reinforcing bar within the top of the beam near the BENF finally fractured, it caused the vertical load resistance to drop to zero. The failure mode of F2 was similar to F1 and thus was not presented.

S2—The measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of S2 is shown in Fig. 7(b). In general, the crack development of S2 was similar to that of S1 and the key points of the test results are listed in Table 2. Thus, only foremost discrepancies between these two specimens were emphasized. For S1, the first diagonal crack in the slab was formed at a load of 22.0 kN (5.0 kips). For S2, however, the diagonal crack in the slab was formed and passed through the center of the slab at a load of 34.0 kN (7.7 kips). For S2, the splitting of the concrete of the joint occurred at a displacement of 260.0 mm (10.2 in.), which was much later than that of S1 due to joint shear reinforcement assistance in maintaining the strength of the diagonal compressive strut by passively confining the joint concrete core. When the vertical displacement reached 69.4 mm (2.7 in.)—that is, equivalent to 3.2% of TDR—the specimen reached the ultimate capacity of 52.0 kN (11.7 kips) and corresponded to 154.0% of the load specified by GSA 3 that is required to resist progressive collapse. When the vertical displacement reached 218.0 mm (8.6 in.), tensile membrane action was observed in the load-displacement curve. Figure 10 illustrates the crack pattern development at different performance levels of the slab of S2.

F3—The measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of F3 is shown in Fig. 7(c). An asymmetrical crack pattern was developed in the longitudinal and transverse beams due to unequal design span in the longitudinal and transverse frames. The first flexural crack was observed in the longitudinal beam at a load of 3.9 kN (0.9 kips). At a load of 10.0 kN (2.3 kips), the first flexural crack occurred in the top of the transverse BENF and the bottom of the longitudinal BENC, respectively. Moreover, the first shear cracks were observed in both
faces of the corner joint at this load stage simultaneously. However, the first flexural cracks at the bottom of the transverse BENAC were observed at a load of 16.0 kN (3.6 kips). When the vertical displacement reached 38.9 mm (1.5 in.), the specimen reached the ultimate capacity of 23.0 kN (5.2 kips) and corresponded to 57.0% of the load specified by GSA that is required to resist progressive collapse.

S3—The measured vertical and horizontal reaction force versus the vertical displacement of the corner joint of Specimen S3 is shown in Fig. 7(c). An asymmetrical crack pattern was developed in the longitudinal and transverse beams due to unequal design span in the longitudinal and transverse frames. The first diagonal crack in the slab was observed at a load of 20.0 kN (4.5 kips). However, the first diagonal crack was not connected to the two interfaces between the beam and the adjacent column. It was similar to an arc and the radius of the arc was the clear span of the short beam. The first flexural cracks occurred in the transverse and longitudinal beam ends near the adjacent columns at the loads of 17.0 and 20.0 kN (3.8 and 4.5 kips), respectively. The first flexural cracks occurring in the BENC, however, were observed at a load of 24.0 kN (5.4 kips) for both beams. The first joint diagonal shear cracks at the face along the transverse and longitudinal directions were observed at the loads of 17.0 and 20.0 kN (3.8 and 4.5 kips), respectively. At a load of 24.0 kN (5.4 kips), new branch cracks were formed to connect the first diagonal crack to the interfaces of the longitudinal and transverse beam-adjacent columns. When the vertical displacement reached 60.2 mm (2.4 in.), the specimen reached the ultimate capacity of 37.5 kN (8.4 kips) and corresponded to 92.8% of the load specified by GSA that is required to resist progressive collapse. The TDR of F2 at PL4 was 5.1%, while that of S2 at PL4 was 6.5%. By comparing the ultimate capacity of F2 and that of S2, it can be seen that F2 and S2 can reach 108.0% and 154.0% of the loads specified by GSA that are required to resist progressive collapse, respectively. Therefore, based on the test results, one can conclude that both of the specimens can survive if one of the ground corner columns was lost due to extreme loads. The initial stiffness of F2 and S2 obtained from the tests was 0.82 and 1.08 kN/mm (4.7 and 6.2 kips/in.), respectively.

For F3 and S3, based on the test data, 57.0% and 92.8% of the loads specified by GSA that are required to resist progressive collapse could be achieved by F3 and S3, respectively. The ultimate capacity of S3 was increased by approximately 63%, compared with F3. Both specimens, however, cannot survive progressive collapse. The initial stiffness of F3 and S3 obtained from the tests was 0.75 and 1.04 kN/mm (5.4 and 6.5 kips/in.), respectively.

It was confirmed that the S-series specimens performed much better compared to the corresponding F-series specimens. The slab functioned as the flange of the L-beam in S-series specimens and significantly increased the flexural capacity of the beam section near the adjacent, enlarged columns. However, it should be noted that the effect of L-beam action on the flexural capacity of the beam end near the corner column is limited due to the fact the slab flange is in the compressive zone.

Resistance capacity decomposition—Figure 12 shows the contribution of the RC slab and beams in the resistant capacity of the S-series specimens in increasing the vertical displacement. Initially, for S1, the RC slab only carried approximately 4.8% of the resistant capacity. With the increasing displacement, the contribution of the slab was increased to 31.2% of the resistant capacity. After that, the slab component reduced to approximately 4.8% due to the first crack forming in the slab. Next, the slab component increased to 38.6% due to the plastic hinge forming in the BENAC. After the displacement reached 200 mm (7.9 in.), the corresponding F-series specimens is shown in Fig. 7. The TDR of F1 at PL4 was 4.2%, while the TDR of S1 at a similar stage was 4.1%. By comparing the ultimate capacity of the two specimens, it can be seen that F1 can reach 77.6% of the load specified by GSA that is required to resist progressive collapse, whereas S1 can reach 116.0% of the required load. Therefore, based on the test results, it can be concluded that S1, but not F1, can survive if the corner column was lost. Moreover, the initial stiffness of F1 and S1 obtained from the tests was 0.82 and 1.08 kN/mm (4.7 and 6.2 kips/in.), respectively.

The TDR of F2 at PL4 was 5.1%, while that of S2 at PL4 was 6.5%. By comparing the ultimate capacity of F2 and that of S2, it can be seen that F2 and S2 can reach 108.0% and 154.0% of the loads specified by GSA that are required to resist progressive collapse, respectively. Therefore, based on the test results, one can conclude that both of the specimens can survive if one of the ground corner columns was lost due to extreme loads. The initial stiffness of F2 and S2 obtained from the tests was 0.95 and 1.13 kN/mm (5.4 and 6.5 kips/in.), respectively.

It was confirmed that the S-series specimens performed much better compared to the corresponding F-series specimens. The slab functioned as the flange of the L-beam in S-series specimens and significantly increased the flexural capacity of the beam section near the adjacent, enlarged columns. However, it should be noted that the effect of L-beam action on the flexural capacity of the beam end near the corner column is limited due to the fact the slab flange is in the compressive zone.

**DISCUSSION OF TEST RESULTS AND SLAB EFFECTS**

**Comparison of performance of F-series specimens to corresponding S-series specimens**

**Load-displacement relationship**—The comparison of the load-displacement relationship of the S-series specimens to the corresponding F-series specimens is shown in Fig. 7. The TDR of F1 at PL4 was 4.2%, while the TDR of S1 at a similar stage was 4.1%. By comparing the ultimate capacity of the two specimens, it can be seen that F1 can reach 77.6% of the load specified by GSA that is required to resist progressive collapse, whereas S1 can reach 116.0% of the required load. Therefore, based on the test results, it can be concluded that S1, but not F1, can survive if the corner column was lost. Moreover, the initial stiffness of F1 and S1 obtained from the tests was 0.82 and 1.08 kN/mm (4.7 and 6.2 kips/in.), respectively.

The TDR of F2 at PL4 was 5.1%, while that of S2 at PL4 was 6.5%. By comparing the ultimate capacity of F2 and that of S2, it can be seen that F2 and S2 can reach 108.0% and 154.0% of the loads specified by GSA that are required to resist progressive collapse, respectively. Therefore, based on the test results, one can conclude that both of the specimens can survive if one of the ground corner columns was lost due to extreme loads. The initial stiffness of F2 and S2 obtained from the tests was 0.95 and 1.13 kN/mm (5.4 and 6.5 kips/in.), respectively.

For F3 and S3, based on the test data, 57.0% and 92.8% of the loads specified by GSA that are required to resist progressive collapse could be achieved by F3 and S3, respectively. The ultimate capacity of S3 was increased by approximately 63%, compared with F3. Both specimens, however, cannot survive progressive collapse. The initial stiffness of F3 and S3 obtained from the tests was 0.75 and 1.04 kN/mm (5.4 and 5.9 kips/in.), respectively.

It was confirmed that the S-series specimens performed much better compared to the corresponding F-series specimens. The slab functioned as the flange of the L-beam in S-series specimens and significantly increased the flexural capacity of the beam section near the adjacent, enlarged columns. However, it should be noted that the effect of L-beam action on the flexural capacity of the beam end near the corner column is limited due to the fact the slab flange is in the compressive zone.

Resistance capacity decomposition—Figure 12 shows the contribution of the RC slab and beams in the resistant capacity of the S-series specimens in increasing the vertical displacement. Initially, for S1, the RC slab only carried approximately 4.8% of the resistant capacity. With the increasing displacement, the contribution of the slab was increased to 31.2% of the resistant capacity. After that, the slab component reduced to approximately 4.8% due to the first crack forming in the slab. Next, the slab component increased to 38.6% due to the plastic hinge forming in the BENAC. After the displacement reached 200 mm (7.9 in.),
the slab component significantly increased due to tensile membrane action developing in the slab.

In general, S2 and S3 have similar behavior as S1. Initially, 5.0% and 5.4% of the resistant capacity was carried by the RC slab for S2 and S3, respectively. When the vertical displacement reached 218.0 and 260.0 mm (8.6 and 10.2 in.) for S2 and S3, respectively, the slab component significantly increased due to tensile membrane action developing in the slab.

Local behavior: reinforcing bar strains—The strains in the reinforcement were measured at several locations in each beam using strain gauges. Figures 13 and 14 illustrate the measured strain distribution of F1 and S1 corresponding to different performance levels, respectively. As shown in Fig. 13, for F1, strain reversal was observed in the beam bottom longitudinal reinforcement in the BENF after removing the ground corner column. The tensile strains of the top longitudinal reinforcement in the BENF significantly increased, while the tensile strains of the bottom longitudinal reinforcement in the BENC started to be reduced after PL3. After PL3, the inflection point (zero strain point) of both the top and bottom longitudinal reinforcements was significantly moving toward the corner joint and this indicated that the resistance mechanism of the specimen was changing to the...
cantilever beams after severe failure occurred in the corner joint. The bottom longitudinal reinforcement in the BENF yielded at PL4, while the bottom longitudinal reinforcement near to the corner joint remained elastic during the test. In general, the strain distribution profile of S1 was similar to that of F1. As illustrated in Fig. 14, the distinguishing point between these two specimens was that the maximum compressive strain of the bottom longitudinal reinforcement in the BENAC of S1 was much larger than that of F1. This may be due to the slab invoking a flange effect on the beam section in S1. The strain distribution profiles for the rest of the specimens were similar and therefore were not presented.

Figure 15 illustrates the relationship of strain in the slab reinforcement versus vertical displacement of S1. The locations of strain gauges are illustrated in Fig. 6. As shown in Fig. 15(a), initially, the ST1 was compressive but it started to decrease when the vertical displacement reached 22.4 mm (0.9 in.). In this vertical displacement, diagonal shear cracks were observed in the corner joint. ST1 converted into tensile after the vertical displacement exceeded 200.0 mm (7.9 in.). Tensile membrane action developed at this displacement stage. The strain of ST3 suddenly increased past its maximum limit when the displacement reached 90.0 mm (3.5 in.). However, the strain of ST6 was close to zero during the test and proved that the majority of the force initially resisted by the damaged corner column was transferred to the adjacent columns, and negligible force was transferred to the interior column.

Figure 15(b) depicts the relationship of the strain in the slab bottom reinforcement versus vertical displacement. The strain in all bottom reinforcements, except SB1 and SB2, were tensile during the test regime. The strains observed in SB3, SB5, and SB8 were much larger than the rest of the strain points due to the fact that major deformation was concentrated in the diagonal crack. When the slab developed a tensile membrane action, the strain distribution in SB5, SB8, SB9, and SB1 indicated that the strain in the slab reinforcement decreased with increasing distance away from the adjacent column. This is one of the major differences between the membrane action developed in the present slabs and that developed in the slabs tested by previous researchers. In their tests, all bottom reinforcements in the center slab were assumed to have yielded after tensile membrane action was developed. Further experimental and analytical studies were needed to analyze this behavior. The strains of SB6 and SB7 were extremely small and was confirmed by the
observed failure mode, with limited damage occurring in the interior half-triangle slab during the test.

FUTURE RESEARCH
To have a deeper understanding of the slab effect on the performance of substructures for progressive collapse caused by losing a corner column, more variables (such as slab reinforcement ratio and slab thickness/span ratio), need to be further investigated in the future. Moreover, the slab effect on the behavior of substructures initiated by losing the exterior and interior column need to be quantified in future studies. Furthermore, analytical studies to capture the tensile membrane action developed in the slabs for progressive collapse should be conducted.

CONCLUSIONS
Based on the experimental study conducted in this research, the following conclusions can be derived:
1. As expected, seismic detailing could significantly improve the ultimate capacity of the substructures. The nonseismically detailed specimens, F1 and F3, only reached 77.6% and 57.0%, respectively, of the loads specified by GSA that are required to resist progressive collapse. However, seismically detailed specimen F2 could reach up to 108.0% of the load specified by GSA that is required to resist progressive collapse and could eliminate the possibility of progressive collapse if the corner column was lost.
2. Experimental results indicated that ignoring the contribution of the RC slab in resisting progressive collapse will be extremely conservative, especially for cast-in-place structures. The ultimate load-carrying capacity of S1, S2, and S3 (with slab) was increased by 51.6%, 40.7%, and 63%, respectively, as compared to Specimens F1, F2, and F3 (without slab).
3. Top beam reinforcement near the fixed end fractured in the final test stage of the F-series specimens, while a penetrating diagonal crack in the slab formed in the final test stage of the S-series specimens. The second ascending branch in the load-displacement curves of the S-series specimens indicated that tensile membrane action was developed in the slab due to limited catenary action, as observed in the F-series specimens. However, further experimental and analytical studies are needed to understand detailed behavior of the membrane action of the slab for progressive collapse under the scenario of the loss of a corner column.

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