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Load-resisting mechanism to mitigate progressive collapse of flat slab structures

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For reinforced concrete flat slab structures, column failure due to unexpected extreme loading may significantly increase the moment and shear force at adjacent slab–column connections, which can trigger punching shear failure at these connections and result in progressive collapse of flat slab structures. To determine the possible load-resisting mechanisms developed in flat slab structures in resisting progressive collapse, three multi-panel flat slab substructures are tested in this study. A flat slab substructure is tested as a control specimen. The remaining two specimens are strengthened by adding drop panels or post-installed shear bolts. Punching shear failure is observed in the control specimen and the specimen with post-installed shear bolts after reaching their yield loads. However, even the control specimen without any strengthening exhibited considerable post-failure resistance and deformation capacity mainly due to sufficient integrity reinforcement, which had been installed in the bottom of the slab. Test results indicated that drop panels not only increase the flexural resistance but also raise the punching shear resistance of the connections significantly. Thus, no obvious punching shear failure is observed in the specimen with drop panels. However, post-installed shear bolts can only increase the punching shear resistance. For post-failure behaviour, this is not very effective. The test results demonstrated that flexural failure dominates the performance of test specimens, as relatively low slab reinforcement ratio but high span/thickness ratio was designed.

Notation

A_s	tensile slab reinforcement per unit width
$\sum A_{sb}$	total area of bottom integrity reinforcement of all principal directions
d_s	slab effective depth
F_{Corner}	vertical load distributed by the corner column
F_{Edge}	vertical load distributed by the edge column
F_T	total vertical load applied on the slab
F_y	yield resistance of test specimen
f'_c	compressive strength of concrete
f_u	ultimate strength of integrity reinforcement
f_y	yield strength of integrity reinforcement
h_1	length of the centre line of segments A
h_2	length of the centre line of segments B
l	column spacing
m_s^P, m_s^N	yield moment of the sagging and hogging yield line
q	external pressure
q_y	pressure corresponding to yield resistance
V_p	residual load-resisting capacity of the slab–column connection
W_E	external work due to applied external load
α, β	measured angles
γ_1	length of hogging yield line in segments A
γ_2	length of hogging yield line in segments B
δ	virtual displacement at the centre column

Introduction

When an initial local failure causes the loss of gravity load capacity of the structure, 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 disproportionate collapse or progressive collapse. To provide necessary design guidelines for practising engineers to evaluate the vulnerability of structures in mitigating progressive collapse, several design guidelines (ASCE–SEI, 2010; COST Action TU0601, 2008; GSA, 2003; US DoD, 2009) had been proposed in North America and the EU. Moreover, in order to refine the design guidelines, several researchers (Astaneh-Asl *et al.*, 2001; Choi and Kim, 2011; Fu, 2010; Gouverneur *et al.*, 2013; Kim and Yu, 2012; Li *et al.*, 2011; Marjanishvili and Agnew, 2006; Qian and Li, 2011, 2012a, 2012b, 2013a, 2013b, 2014; Qian *et al.*, 2014a, 2014b; Su *et al.*, 2009; Yap and Li, 2011; Yu and Tan, 2014) have investigated the ability of reinforced concrete (RC) moment frames in resisting progressive collapse by numerical or experimental approaches. However, relatively little attention has been paid to flat slab structures in resisting progressive collapse. The collapse of Sampoong department store, Seoul, South Korea indicated that flat slab structures have much higher vulnerability compared to moment frames. When one of the columns is lost due to unexpected extreme load, load redistribution will result in significantly increased bending moment and shear force at adjacent slab–column connections,

causing these connections to fail, and resulting in progressive collapse of the structure. Existing design guidelines (US DoD, 2009) addressed the design approach and acceptance criteria for flat slab or plate structures to resist progressive collapse. However, the acceptance criteria are based on the seismic evaluation and rehabilitation guidelines (ASCE 41-06 (ASCE–SEI, 2007)). As the scenario of a missing column is different to the lateral cyclic earthquake load condition, using the acceptance criteria obtained from seismic tests to extrapolate the inelastic deformation capacity of flat slabs resisting progressive collapse is controversial. Mitchell and Cook (1984) discussed the possibility of developing compressive membrane and tensile membrane action in flat slab structures to mitigate progressive collapse by analytical prediction. However, relatively few tests have been conducted to evaluate the reliability of developing compressive membrane and tensile membrane action in flat slab or plate structures to resist progressive collapse. Moreover, it is necessary to quantify the acceptance criteria of flat slab structures to resist progressive collapse and to refine existing design guidelines. Therefore, three four-panel flat slab substructures of size 3750 mm × 3750 mm (12.3 ft × 12.3 ft) were tested in Nanyang Technological University, Singapore for this purpose. Tested specimens were at one-quarter scale owing to

space limitations in the laboratory. One flat slab substructure was tested and taken as the control specimen. The remaining two specimens were punching shear strengthened by adding drop panels or post-installed shear bolts.

Description of test programme

Experimental substructures

The non-seismically designed prototype RC flat slabs (WD, ND and ND-P) were designed according to code ACI 318-11 (ACI, 2011). WD, ND and ND-P represent a flat slab with drop panel, flat slab without drop panel and flat slab without drop panel but strengthened by post-installed shear bolts, respectively. The dead load of the prototype structure due to the 220 mm (8.7 in) thick slab was 5.2 kPa (108.7 psf). The additional dead load was assumed to be 1.0 kPa (20.9 psf). The live load was assumed to be 3.0 kPa (62.7 psf). An additional uniform pressure of 1.8 kPa (37.6 psf) was assumed to simulate the weights from infill walls. One-quarter scaled substructures with size of 3750 mm × 3750 mm × 55 mm (12.3 ft × 12.3 ft × 2.2 in) were cast and tested in this study. Figure 1 illustrates the dimensions and reinforcement details of specimen WD. As shown in the figure, the specimen has one

Figure 1. Details of specimen WD (unit: mm) (note: 1 mm = 0.0393 in; T = deformation reinforcing bar; R = plain reinforcing bar)

interior column, eight surrounding columns, nine 35 mm (1.4 in) thick drop panels and a 55 mm (2.1 in) thick RC slab cast monolithically. The dimension of column is enlarged to 200 mm (7.9 in) square to ensure failure would not occur in these enlarged columns and equivalent fixed boundary conditions could be applied on the slab. Moreover, the interior and surrounding columns were respectively reinforced by 4-T13 and 8-T13 to further enhance the strength and stiffness of supporting columns. The dimensions of the drop panel are 450 mm (17.7 in) square and single-layer reinforced by R6@80 mm (R6@3.1 in). R6 and T13 represent plain reinforcement with diameter 6 mm (0.2 in) and deformed reinforcement with diameter 13 mm (0.5 in), respectively. The concrete cover of the column and slab was 20 mm and 7 mm (0.8 in and 0.3 in), respectively. The slab was reinforced by two layers of reinforcement (R6) with spacing of 250 mm (9.8 in). As the diameter of the slab reinforcement could not be scaled down proportionally due to rebar varietal limiting, the spacings of slab reinforcement were relatively large to keep a slab reinforcement ratio similar to the prototype structures. Integrity reinforcement was installed in the bottom layer, which was continual and passed within the column cages. In this study, the amount of integrity reinforcement was 3R6 in each principal direction, which was larger than the requirement suggested in ACI 318-11 (ACI, 2011) (no specific calculation but required two bottom bars passing through the column for integrity reinforcement). For specimens ND and ND-P, the dimensions and reinforcement details were similar to specimen WD, but without drop panels. Comparing with specimen ND, a series of bolts with diameter of 6 mm (0.2 in) were post-installed in the slab-column connections of specimen ND-P to study the effects of post-installed bolts on progressive collapse behaviour of flat slabs. As shown in Figure 2, the bolts in combination with epoxy were used to install punching shear reinforcement into already hardened concrete. The spacing of bolts was 35 mm (1.4 in), which is about $0.75d$, where d was the effective depth of the slab. The length of bolts was 60 mm (2.4 in); 45 mm (1.8 in) of the bolts were inserted from the top slab for convenience. Figure 3 shows the arrangement of strengthening bolts in different columns. The yield strength and ultimate strength of $\Phi 6$ bolt is 326 and 407 MPa (6.8×10^6 , 8.5×10^6 psf), respectively. The dimensions and reinforcement details of specimens are shown in Table 1. The measured concrete compressive strengths of specimens ND, WD and ND-P at 28 d after casting are 25.2, 22.3 and 24.1 MPa (5.3×10^5 , 4.7×10^5 and 5.0×10^5 psf), respectively. The yield strengths of R6 and T13 are 500 and 529 MPa (10.4×10^6 and 11.0×10^6 psf), while the ultimate strengths of R6 and T13 are 617 and 608 MPa (12.9×10^6 and 12.7×10^6 psf).

Figure 2. Detailing of post-installed shear bolts in specimen ND-P

Figure 3. Arrangement of strengthening bolts in specimen ND-P: (a) edge column; (b) corner column

Design of test set-up

A scheme of the test set-up of a typical test specimen is shown in Figure 4. To study the performance of flat slab structures under the loss of an interior column scenario, the interior (centre) column was assumed to have been removed before applying vertical load on the slab. Eight strong steel legs were utilised to support the flat slab. Each steel leg was connected with a 40 mm (1.6 in) thick strong plate by four $\Phi 27$ bolts, while the steel plates were fastened to the strong floor by way of pre-tensioned steel rods with a diameter of 60 mm (2.4 in). Moreover, eight steel plates hung below the extending part of the slab to simulate

Test	Elements					Slab rebar Top layer: mm	Slab rebar Bottom layer: mm
	Interior column stub	Edge or corner columns	Drop panel thickness	Drop panel rebar: mm	Slab thickness: mm		
ND	Height = 390 mm Cross-section = 200 × 200 mm ² Reinforcement ratio = 1.3%	Height = 300 mm Cross-section = 200 × 200 mm ² Reinforcement ratio = 2.6%	NA	NA	55	R6@250	R6@250
WD			35 mm	R6@80	55	R6@250	R6@250
ND-P			NA	NA	55	R6@250	R6@250

Note: 1 mm = 0.0393 in; NA indicates not applicable.

Table 1. Specimen properties

Figure 4. Overview of a specimen in position ready for testing
(note: 1 mm = 0.0393 in)

the influence of continuity of the surrounding slab on the response of test specimens. A hydraulic jack with a stroke of 600 mm (23.6 in) was utilised to apply load on the slab. In order to simulate uniformly distributed load (UDL) applied on the slab, a special load distribution tree (item 9 in Figure 4) was designed. Figure 5 presents the detailing of the load distribution tree. As shown in Figure 5(a), the load distribution tree consisted of three rigid beams and four triangular steel plates. The secondary steel beams were connected with the steel plates by hemispherical ball-and-socket joints to ensure that the applied load remained vertical even if the slab suffered relatively large deformation. It had been proved that the proposed load approach could equivalently simulate uniform pressure applied on the specimen by way of static finite-element analysis (Diana, 2008). Moreover, in order to make sure that the applied concentrated load from the

hydraulic jack kept vertical and to obtain symmetric failure modes, another steel assembly (item 4 in Figure 4) was designed. This steel assembly consists of a steel box with four high-stiffness pins in each direction. Thus, the steel column (item 3 in Figure 4) only moves freely in the vertical direction and horizontal freedom was partially restrained.

Instrumentation

A large number of measurement devices were installed both internally and externally to monitor the response of the test specimens. A load cell (item 1 in Figure 4) was utilised to measure the vertical load applied on the loading distribution tree. Three tensile and compressive load cells (RC1, RC2 and RC3 in Figure 5(b)) were installed in three of the rigid steel legs to measure the vertical force redistribution. A series of linear variable deformation transducers (LVDTs) and linear potentiometers were also placed at various locations on the substructure to measure the different types of internal deformation. As test specimens are symmetrical in both directions, only one panel of the slab was monitored by 16 strain gauges, as shown in Figure 6.

Experimental results

Global behaviour

Control specimen ND: The measured load–displacement curve of ND is shown in Figure 7(a). The first top cracks were observed at interfaces between the edge column and slab at a load of 63.5 kN (14.3 kip). When the load reached 82.9 kN (18.7 kip), the cracks developed and connected as a circle. At a load of 92.9 kN (20.9 kip), more cracks occurred at edge columns, which radiated away from the column periphery, as shown in Figure 8. When the load reached 116.9 kN (26.3 kip), the top reinforcement near the adjacent column yielded. For a typical, isolated, slab–column connection test, which is a popular simplification model to study punching shear resistance of slab–column connections, the test could stop as the specimen reached flexure failure. However, investigating the possible load-resisting mechanisms to resist progressive collapse of flat slab structures was the main concern in this study. Thus, the vertical displacement was further increased and the test was continued. When the load reached 154.7 kN (34.8 kip), secondary punching failure occurred at one of the edge slab–column connections. Then, more loads were transferred into intact edge columns. At a load of 163.4 kN (36.8 kip), secondary punching failure occurred at three of the edge slab–column connections and load resistance dropped drastically. The term of ‘secondary punching failure’ was utilised herein owing to the punching shear failure being observed after reaching yielding load and to distinguish the ‘primary punching failure’, which is the main failure mode of isolated slab–column connections tests. Thus, comparing to the yield load, the first peak load (FPL) was increased by 39.8%. With further increase of vertical displacement, more loads have to be transferred into corner columns. When vertical displacement reached 75 mm (3.0 in), secondary punching shear failure was observed not only in the edge slab–column connections but also in the corner slab–column

(a)

(b)

Figure 5. Detailing of load distribution tree (item 9 in Figure 4):
(a) schematic diagram of load distribution by way of loading tree;
(b) arrangement of 12 loading points (note: 1 mm = 0.0393 in)

Figure 6. Strain gauge locations: (a) top layer; (b) bottom layer (note: ST, SB represent top and bottom of slab, respectively; 1 mm = 0.0393 in)

connections. Moreover, as shown in the load–displacement curve, the load resistance did not keep reducing with increasing vertical displacement. It began to re-ascend at a certain displacement stage. This behaviour is called post-failure performance and is very valuable for structures in resisting unpredictable progressive collapse risks. When the displacement of the interior column reached 160.2 mm (6.3 in), the ultimate load capacity (ULC) of 188.9 kN (42.5 kip), which is about 115.6% of its FPL, was

Figure 7. Comparison of measured load–displacement curve with analytical prediction of test specimens: (a) specimen ND; (b) specimen WD; (c) specimen ND-P (note: IL, MYL, FPL, and ULC represent initial crack load, predicted yield load, measured yield load, first peak load and ultimate load capacity, respectively)

achieved. ULC herein is defined as the maximum load capacity measured at the large displacement stage. The high post-failure resistance may be attributed to two components: dowel action of integrity reinforcement and secondary load-carrying mechanisms (one-way catenary or two-way tensile membrane action) developed in the slab, which will be further discussed in later sections. The failure mode of ND is shown in Figure 9. As shown in the figure, severe secondary punching failure was observed in edge and corner connections. Despite flexural circular cracks being observed at the top face of the slab, their crack width was not very wide. Looking at the deformation shape of this specimen,

Figure 8. Crack pattern of specimen ND: (a) top view; (b) bottom view

the majority of the deformation is concentrated around the columns.

Strengthened specimen WD: WD had similar dimensions and reinforcement details to ND, with punching shear strengthened by adding drop panels at the column head. The measured load–displacement curve of WD is shown in Figure 7(b). In general, the crack development of WD is similar to that of ND; the key

Figure 9. Failure mode of specimen ND

results are listed in Table 2. Thus, only the most conspicuous discrepancies between these two specimens were emphasised. Similar to ND, the first crack developed in the interface between slab and edge column. However, the widest flexural cracks were formed at the edges of drop panels. As expected, despite several cracks radiating away from the column periphery also occurring in WD, no obvious punching failure was observed during the test, mainly owing to the drop panel, which had increased the punching shear resistance significantly. Moreover, when the load reached 224.1 kN (50.5 kip), the load resistance began to drop owing to greater width of cracks and concrete crushing at the slab–drop panel interfaces. However, the decrease in load resistance of WD is much milder than for ND as its failure was controlled by flexural damage and it was more ductile. Similar to ND, load resistance re-ascending was also observed in WD. The measured ULC at the large displacement stage was 233.9 kN (52.7 kip), which is about 104.4 % of its FPL. Figures 10 and 11 show the crack pattern and failure mode of WD, respectively. As shown in the figures, no punching failure occurred in this specimen and major damage was concentrated in the circular crack, which connected with the flexural cracks at the edges of drop panels. Compared to ND, the width of the flexural cracks is much greater and the deformation of the slab is milder.

Strengthened specimen ND-P: ND-P had similar dimensions and reinforcement details to ND but was strengthened by post-installing shear bolts near the columns. The measured load–displacement curve of ND-P is shown in Figure 7(c). The first cracks occurred at the interface between the slab and edge column at a load of 48.0 kN (10.8 kip), which is significantly lower than the first crack load of ND. This is possibly due to micro-cracks generated as a result of post-drilling holes for strengthening. Moreover, for this specimen, cracks radiating away from the column periphery occurred before the circular crack formed. When the displacement reached 30.3 mm (1.2 in) and load reached 121.3 kN (27.3 kip), yielding was observed in some

Test	Initial crack load, IL: kN (kip)	Measured yield load, MYC: kN (kip)	Initial stiffness: ^a kN/mm (kip/in)	First peak load, FPL: kN (kip)	Displacement at FPL: mm (in)	Ultimate load capacity, ULC: kN (kip)	Maximum displacement: mm (in)
ND	63.5 (14.3)	116.9 (26.3)	3.9 (22.0)	163.4 (36.8)	65.2 (2.6)	188.9 (42.5)	160.2 (6.3)
WD	70.3 (15.8)	168.2 (37.9)	4.4 (25.1)	224.1 (50.5)	80.2 (3.2)	233.9 (52.7)	159.7 (6.3)
ND-P	48.0 (10.8)	121.3 (27.3)	4.0 (22.8)	190.2 (42.8)	64.9 (2.6)	196.0 (44.1)	165.0 (6.5)

^a Initial stiffness = secant stiffness at yield resistance.

Table 2. Test results

of the slab reinforcement, which was close to that of ND. Similarly to ND, with strain hardening and the development of compressive membrane action, load resistance kept increasing after the reinforcement had yielded. Secondary punching failure was first at the load of 175.1 kN (39.4 kip). Similarly to ND, the load kept increasing until secondary punching shear failure was observed at all connections. The measured FPL of ND-P is 190.2 kN (42.8 kip). Although both ND and ND-P suffered secondary punching shear failure, as expected, the perimeter of the critical section of the punching shear of ND-P is much larger than that of ND. Similarly, considerable post-failure resistance and deformation capacities were observed in this specimen. The measured ULC was 196.0 kN (44.1 kip), which is 103.0% of its FPL. Figures 12 and 13 show the crack pattern and failure mode of ND-P, respectively. As shown in the figures, severe secondary punching shear failure occurred at slab-column connections, especially for edge columns. Moreover, the critical section of punching shear of the connection is beyond the edge of post-installed bolts. However, similarly to ND, the majority of the deformation is concentrated surrounding the columns at the final stage of the test.

Strain gauge results

As shown in Figure 6, 'ST' and 'SB' are the labels of strain gauges attached in the top and bottom layer reinforcement, respectively. Figure 14 illustrates the strain gauge reading measured from the slab reinforcement of ND. As shown in the figure, the first yield is measured in ST3 due to the majority of the load being redistributed into the adjacent edge columns and the main flexural crack passing through the interface between the slab and edge column. Conversely, limited tensile strain is measured in ST4 because only about 7% of the force redistributes into the corner column and the main flexural crack is far away from the interface between the slab and corner column. Moreover, considerable tensile strain is recorded in SB1, SB2 and SB3 owing to reversal of the bending moment. Comparing the values of SB1 with SB2, it is found that SB1 exceeded the yield strain significantly, whereas SB2 did not, although both of them keep increasing with increase in vertical displacement. This can be attributed to stronger anchorage in the integrity reinforcement (SB1) compared to the normal bottom reinforcement (SB2), even

in the column strip. The values of the remaining bottom strain gauges (SB6, SB9 and SB12) kept increasing with increase of the vertical displacement, possibly owing to the cracks at the bottom face, which become wider and wider. Moreover, the bottom slab reinforcement performed like a tensile net when tensile membrane action began to develop in the slab at the large displacement stage. In general, WD had similar strain gauge results to those of ND, except SB1 and SB2. In WD, both SB1 and SB2 could reach their yield strain, possibly because the drop panel could provide an additional anchor to SB2.

Load redistribution response

Structures must find an alternative load path to redistribute the force initially resisted by the lost column. If no effective load path can be generated, the structures will undergo progressive collapse. Thus, it is necessary to monitor the load redistribution response of the specimens. As mentioned in the section on 'Instrumentation', three tension/compression load cells (RC1, RC2 and RC3) were installed in three of the corner supports to monitor the load redistribution before tests. Figure 15 shows the percentage of vertical load transfer into corner columns with increasing vertical displacement at the interior column of ND. As the result of RC3 is unstable, only the results of RC1 and RC2 are presented. For the locations of RC1, RC2 and RC3, refer to Figure 5(b). As shown in the figure, about 6.9% of the vertical load was transferred into the corner column when secondary punching shear failure had occurred in all of the edge slab-column connections. After that, more loads were transferred into corner columns until secondary punching shear failure had also formed in the corner column. At the final stage of the test, about 7.4% of the load was transferred into the corner column. For ND-P, about 7.9% and 7.3% of the load was transferred into the corner column when secondary punching shear failure had occurred at all edge slab-column connections. However, for WD, the percentage of load distribution into the corner column keeps increasing with increase of the vertical displacement. At first peak resistance and final stage of the test, about 7.1% and 8.3% of the load was transferred into the corner column of this specimen, respectively. It should be noted that the load transferring into edge columns was determined by assuming that the failures occurring in edge columns are simultaneous and using Equation 1.

Figure 10. Crack pattern of specimen WD: (a) top view; (b) bottom view

$$1. \quad F_{\text{Edge}} = \frac{F_T}{4} - F_{\text{Corner}}$$

where F_{Edge} is the vertical load distributed by the edge column; F_T is the total vertical load applied on the slab; and F_{Corner} is the vertical load distributed by the corner column.

Figure 11. Failure mode of specimen WD

Acceptance criteria

As listed in table 4-3 of US DoD (2009), the acceptance criteria for slab–column connections with continuity reinforcement is 0.05 rad of chord rotation, which is defined as the ratio of vertical displacement in the centre column to the column spacing of the flat slab structures. However, the measured acceptance criteria in specimens ND, WD and ND-P are 0.12, 0.12 and 0.13 rad, respectively. Thus, the acceptance criteria shown in US DoD (2009) for flat slab structures are over-conservative. As predicted, the main reason for these over-conservative values is that the acceptance criteria listed in US DoD (2009) are adopted from the criteria shown in guideline ASCE 41-06 (ASCE–SEI, 2007), which is mainly for seismic evaluation and rehabilitation. More tests with varying details should be conducted to generate a data pool of acceptance criteria for flat slab structures under progressive collapse scenarios in future.

Discussion of test results

Flexural resistance

Failure observations during tests and strain gauge readings indicated that the failure of test specimens was dominated by flexural failure, although severe secondary punching shear failure occurred in ND and ND-P after reaching their flexural resistance. To evaluate the effectiveness of yield-line approaches for estimating flexural capacity of multi-panel flat slab or flat plate structures, analytical yield-line analyses were carried out. The flexural capacity of test specimens is determined by way of the principle of virtual work, based on the observed yield-line configuration, as shown in Figure 16. It should be noted that, for WD, the hogging yield line connected with the edges of the drop panels. The internal virtual work due to rotation of the yield lines is

$$2. \quad \sum W_1 = 4\delta \left[m_s^N \left(\frac{\gamma_1}{h_1} + \frac{\gamma_2}{h_2} \right) + 2m_s^P \left(\frac{h_1}{h_2} \sin \frac{\partial}{2} + \sin \frac{\beta}{2} \right) \right]$$

Figure 12. Crack pattern of specimen ND-P: (a) top view; (b) bottom view

where W_I is the internal work due to strain energy; δ is virtual displacement at the centre column; h_1 and h_2 are the lengths of the centre-lines of segments A and B, respectively; γ_1 and γ_2 are the lengths of hogging yield lines in segments A and B, respectively; the measured angles of α and β are 69° and 21° , respectively; m_s^P and m_s^N are the yield moment of the sagging and hogging yield lines, respectively, which can be determined by Equation 3.

Figure 13. Failure mode of specimen ND-P

Figure 14. Strain gauge reading measured in slab reinforcement of specimen ND

Figure 15. Percentage of vertical load transfer into corner columns with varying of vertical displacement of interior column of specimen ND

--- Hogging yield line — Sagging yield line
(a)

--- Hogging yield line — Sagging yield line
(b)

Figure 16. Yield line patterns considered to determine flexural resistance of specimens: (a) specimens ND and ND-P; (b) specimen WP

$$3. \quad m_s = A_s f_y \left(d_s - 0.59 A_s \frac{f_y}{f'_c} \right)$$

where A_s is the tensile slab reinforcement per unit width and d_s is the slab effective depth.

The external virtual work is

$$4. \quad \sum W_E = q \frac{2\delta}{3} (r_1 h_1 + r_2 h_2)$$

where W_E is the external work due to applied external load and q is the external pressure.

The predicted yield pressures of specimens ND, WD and ND-P are 14.1 kPa (294.7 psf), 20.9 kPa (436.8 psf) and 14.1 kPa (294.7 psf), respectively. Thus, the predicted yield resistances of specimens ND, WD and ND-P, respectively, are 126.9 kN (28.8 kip), 188.1 kN (42.4 kip) and 126.9 kN (28.8 kip) according to Equation 5. However, the measured yield resistances of specimens ND, WD and ND-P are 116.9 kN (26.3 kip), 168.2 kN (37.9 kip) and 121.3 kN (27.3 kip), respectively. Thus, they are matched well.

$$5. \quad F_y = q_y \times (2l)^2$$

where F_y is the yield resistance of test specimen, q_y is the pressure corresponding to yield resistance and l is the column spacing.

Compressive membrane action

Both test results and yield-line analysis indicated that the FPL of test specimens is significantly larger than their yield load capacities, although severe secondary punching shear failure was observed in the slab-column connections of ND and ND-P. As shown in Figure 7, the measured FPLs of ND, WD and ND-P are 163.9 kN (36.8 kip), 224.1 kN (50.5 kip) and 190.2 kN (42.8 kip), which enhanced their yield load by 39.8%, 33.2% and 56.8%, respectively. This can be attributed to the strain hardening of the reinforcement and compressive membrane action (CMA) developed in the RC slab. It should be noted that above yield load, which was predicted by yield-line analysis, is based on the measured yield strength of R6 (500 MPa (10.4×10^6 psf)). If the ultimate strength 617 MPa (12.9×10^6 psf) is utilised for yield-line analysis, the yield load could increase by 23.4%, which is still less than the measured enhancement factor. This indicates the existence of CMA to further enhance the load-resisting capacity of test specimens. As three-dimensional tests were conducted, it was difficult to measure the in-plane force developed in the slab quantitatively. However, the horizontal movement of the adjacent supports may provide the evidence of developing membrane actions in the slab. Figure 17 shows the horizontal movement of one of the adjacent columns. For ND, the outward (negative) movement was observed after circular cracks occurred at the slabs and reached a resistance load of 86 kN (19.5 kip). The maximum outward movement of ND is -0.35 mm (0.01 in), corresponding to the vertical displacement of 60.2 mm (2.37 in). When the vertical displacement reached beyond 67 mm (2.63 in), outward movement began to drop due to secondary punching shear

Figure 17. Comparison of horizontal movement response at adjacent support of test specimens

occurring at the adjacent slab–column connections. On further increase of the vertical displacement, outward displacement transferred to inward and indicated that the CMA transferred to tensile membrane action (TMA), which will be discussed in the next section. For WD, the outward movement becomes milder after reaching a maximum value of -0.5 mm due to concrete crushing, which became severe at the edge of the drop panels. For ND-P, a similar response was observed.

Dowel action and tensile membrane action (TMA)

Typically, tests of flat slab specimens are terminated shortly after the load-resisting capacity begins to drop, which may be caused by local punching shear failure or flexural failure. However, as mentioned above, the post-failure behaviour of slab–column connections is very important in preventing the propagation of local punching shear failure throughout the entire floor. Thus, the tests were continued with further increase in vertical displacements to measure the residual load-resisting capacity and deformation capacity.

As shown in Figure 7, considerable residual load-resisting capacity and deformation capacity was observed in the test specimens. As discussed by Mitchell (1993), the dowel action from bottom integrity reinforcement and TMA from remaining

bottom slab reinforcements may provide some residual gravity load-resisting capacity. To disassociate the contributions from dowel action and TMA, existing design equations for dowel action were utilised. Table 3 gives a comparison of the measured ultimate load capacity with the residual load-resisting capacity predicted by design equations or suggestions from existing design codes and previous research studies. The design equations recommended by existing design codes and previous research studies are illustrated in the appendix. As shown in Table 3, the design equations recommended by CSA (2004), Melo and Regan (1998) and Mitchell (1993) will acquire conservative results. This is because only the integrity of reinforcement was considered to provide residual load-resisting capacity. However, the bottom reinforcement, which is beyond the column cage but within the column strip, could also provide additional residual load-resisting capacity by dowel action, as shown in Figure 18. Moreover, TMA developing in the bottom reinforcement could further improve the post-failure behaviour, as shown in Figure 19. Therefore, the realistic multi-panel flat slabs could develop much higher residual load-resisting capacity than the values predicted by design equations, which were derived based on isolated slab–column connections. However, more studies should be carried out to theoretically separate the contribution of dowel action developed in the integrity reinforcement and the tensile membrane action

Figure 18. Schematic diagram of dowel action of integrity reinforcement and bottom reinforcement within drop panel of specimen WD

Test	$V_{Test}^{.b}$ kN (kip)	$V_{CSA}^{Predicted.c}$ kN (kip)	$V_{Mitchell}^{Predicted.c}$ kN (kip)	$V_{Melo\&Regan}^{Predicted.c}$ kN (kip)	$\frac{V_{Test}}{V_{CSA}^{Predicted}}$	$\frac{V_{Test}}{V_{Mitchell}^{Predicted}}$	$\frac{V_{Test}}{V_{Melo\&Regan}^{Predicted}}$
ND-Adj. ^a	31.9 (7.2)	21.2 (4.8)	21.2 (4.8)	21.4 (4.8)	1.50	1.50	1.49
WD-Adj.	39.1 (8.8)	21.2 (4.8)	21.2 (4.8)	21.4 (4.8)	1.84	1.84	1.83
ND-P-Adj.	34.3 (7.7)	21.2 (4.8)	21.2 (4.8)	21.4 (4.8)	1.62	1.62	1.60

^a ND-Adj. is the residual load-resisting capacity of adjacent slab–column connection of specimen ND.
^b V_{Test} is the measured post-failure resistance of edge slab–column connection by assuming failure occurring in edge column is simultaneous.
^c $V_{CSA}^{Predicted}$, $V_{Mitchell}^{Predicted}$, $V_{Melo\&Regan}^{Predicted}$ represent the predicted post-failure resistance according to the codes of CSA (2004), Mitchell (1993) and Melo and Regan (1998), respectively.

Table 3. Comparison of the measured post-failure resistance with the design codes

Figure 19. Schematic diagram of tensile membrane actions developed in bottom slab reinforcement

developed in the remaining bottom reinforcement. As shown in Figure 17, the inward movement was measured in the adjacent columns at the large displacement stage. The measured maximum inward movements of ND, WD and ND-P are 0.35 mm (0.01 in), 0.35 mm (0.01 in) and 0.4 mm (0.02 in), respectively. For WD, the large residual resisting capacity of this specimen at the large displacement stage is mainly attributed to the membrane action which developed in the bottom reinforcement.

Conclusions

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

- (a) The propagation of secondary punching shear failure from adjacent slab–column connections to corner slab–column connections demonstrated the high likelihood of progressive collapse of flat slab structures.
- (b) The failure modes of control flat plate specimen ND consisted of severe secondary punching shear failure along with rupture of the slab reinforcements. The occurrence of secondary punching shear failure is mainly because the load-resisting capacity kept increasing after the yield load had been achieved; this was due to strain hardening of the reinforcement and the development of considerable in-plane compressive force in the slab.
- (c) Considerable residual load-resisting capacity and deformation capacity were observed in specimen ND, mainly because the continuous integrity reinforcement passing through the column cages could develop dowel action and the remaining bottom reinforcement could develop tensile membrane actions.
- (d) Drop panels could significantly increase the punching shear resistance of slab–column connections and prevent the occurrence of secondary punching shear failure at the slab–column connections. Post-installed bolts could also significantly increase the punching shear resistance of slab–column connections, but secondary punching shear failure still occurred after reaching the yield load-resistant capacity. Moreover, post-installed bolts may jeopardise the initial stiffness of flat slabs and could not improve the post-failure performance of flat slabs effectively. This is mainly because the residual load-resisting capacity is largely attributed to the

dowel action of integrity reinforcement and tensile membrane action of the remaining bottom reinforcement.

Appendix

The design equations proposed by existing design codes and previous research studies are shown below.

CSA standard (CSA, 2004)

$$6. \quad V_P = 0.5 \times \sum A_{sb} \times f_y$$

where V_P is the residual load-resisting capacity of the slab–column connection; $\sum A_{sb}$ is the total area of the bottom integrity reinforcement for all principal directions; and f_y is the yield strength of integrity reinforcement.

It should be clarified that, in this study, $\sum A_{sb}$ only considers the area of integrity reinforcement placed over the edge columns and connected with the interior column for test specimens. Moreover, it was assumed that, after punching shear failure, the structural integrity reinforcement is capable of yielding and forms an angle of 30° from the horizontal.

ACI 318-11 (ACI, 2011)

ACI 318-11 (ACI, 2011) requires that the bottom reinforcement in the column strip be lapped with Class B tension splices and that at least two bottom reinforcements must pass within the region bonded by the longitudinal reinforcement of the column (integrity reinforcement) in each direction and shall be anchored at exterior supports. Thus, ACI 318-11 (ACI, 2011) did not pay enough attention to specifying the minimum integrity reinforcement to ensure the development of sufficient residual load-resisting capacity of flat slabs and to reduce their vulnerability to disproportional collapse.

Mitchell (1993)

The equation proposed by Mitchell (1993) is adopted by the design code of the CSA standard (CSA, 2004). Thus, it is similar to Equation 6.

Melo and Regan (1998)

Melo and Regan (1998) proposed that the fracture resistance be determined as

$$7. \quad V_P = 0.44 \sum A_{sb} \times f_u$$

where f_u is ultimate strength of integrity reinforcement;
 $f_u = 1.15f_y$ when a reading of f_u is unavailable.

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