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Himawan, Aris; Teng, Susanto

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Cyclic Behavior of Post-Tensioned Slab-Rectangular Column Connections

by Aris Himawan and Susanto Teng

This paper describes testing of three interior post-tensioned slab-column connections having rectangular columns with a column aspect ratio of 5. The first specimen was tested under gravity load only. The second specimen was subjected to gravity and unidirectional cyclic lateral loading, and the third specimen was subjected to gravity and bidirectional cyclic lateral loading. It was confirmed that the strength and stiffness of the connections are much higher along the strong column direction than those along the weak column direction. The test results also indicate that bidirectional cyclic lateral loading significantly reduces the strength, drift capacity, ductility, and stiffness of the connections. The increase in shear strength due to prestressing for connections having rectangular columns tends to be lower compared with those having circular/square columns. Furthermore, a modification to the ACI shear strength equation is proposed to make the prediction conservative for connections having rectangular columns.

Keywords: bidirectional loading; cyclic loading; flat plate; post-tensioning; prestressed concrete slabs; punching shear; rectangular columns; slab-column connections.

INTRODUCTION

The major shortcoming of flat plate structure is its vulnerability to a brittle punching shear failure at its slab-column connections. This kind of failure occurs without warning and may lead to progressive collapse of the structure. At a slab-column connection, shear failure may occur due to transfer of shear forces and unbalanced moments between the slab and the column. The shear transfer usually comes from the gravity load whereas the unbalanced moment may come from different loading between the adjacent spans, or lateral loading, such as strong wind or earthquake loading. For slab-column connections subjected to earthquake loading, the effect of cyclic loading may cause shear strength degradation that can lead to premature failure.

One of the main factors affecting shear strength is column rectangularity. For the same perimeter of critical section, rectangular geometry leads to lower shear strength of slabs compared with square perimeter due to the stress concentration around the shorter side of the critical section. Unfortunately, past research on post-tensioned slab-column connections was limited to connections with square columns, and to the authors' knowledge, no experimental data are available on post-tensioned slab-column connections with rectangular columns. The use of rectangular columns itself is very popular in high-rise apartment or residential buildings, due to architectural detailing that frequently requires a column to be flush with adjacent walls.

RESEARCH SIGNIFICANCE

This paper describes the testing of three interior post-tensioned slab-column connections having rectangular columns with a column aspect ratio of 5 subjected to gravity and cyclic lateral loading. The effects of prestressing, column rectangularity, and unidirectional and bidirectional cyclic lateral loading were studied. The behavior of the connections in terms of strength, stiffness, drift capacity, and ductility was also investigated. It was found that the increase in shear strength due to prestressing for connections having rectangular columns is much smaller compared to those with circular/square columns. In addition, comparisons of the experimental results with predictions of ACI 318-11 (ACI Committee 318 2011) are also presented.

EXPERIMENTAL PROGRAM

Specimens

The specimens represent interior slab-column connections, taken arbitrarily at the fifth story of the model structure. The slab was terminated around the lines of contraflexure under gravity loading, whereas the column was terminated at the middle of the story height. For slab-column connections under gravity loads combined with unbalanced moments, the contraflexure lines would move slightly away from the usual location, but it would not be at the midspan. The specimen dimensions were then adjusted to fit into the testing frame, resulting in a slab dimension of 2540 x 3500 mm (100.0 x 137.8 in) and a column height of 950 and 1150 mm (37.4 and 45.3 in.) above and below the slab surface, respectively.

The specimen configuration leads to six uniformly distributed tendons in an x -direction (shorter direction) and 12 banded tendons in a y -direction (longer direction). To obtain the same compressive stress as in the model structure, the effective forces per tendon of the specimen were then adjusted. At the slab edges, anchorage reinforcements ($4\phi 10$ and $\phi 10@100$, or 10 mm deformed bars at 100 mm center-to-center spacing) were provided to prevent splitting of concrete due to prestressing. This detail of reinforcement is similar to the requirement of ACI 423.3R-96 (ACI Committee 423 1996). Figure 1 shows the layout and profile of the slab tendons. The top and bottom slab reinforcements were provided as in the model structure, whereas the column reinforcements were designed such that the slab,

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Fig. 1—Layout and profile of slab tendons.

Fig. 2—Location of slab reinforcements and strain gauges.

rather than the column, would fail first. The details of the slab reinforcements are depicted in Fig. 2.

Materials

The specimens were cast using ready mixed concrete having a specified cube strength f_{cu} of 40 MPa (5800 psi) or specified cylinder strength f'_c of 32 MPa (4600 psi). All the reinforcing bars used in this experimental program were from high-strength deformed steel bars (T-bars). Monostrand unbonded tendons were used as the slab main reinforcements. These tendons consist of strands coated with grease and extruded in plastic sheathings to prevent bonding and corrosion. The plastic sheathings were made

from polyethylene or polypropylene with approximately a 1 mm (0.04 in.) wall thickness and an 18 mm (0.71 in.) outside diameter. The strands were 13 mm (0.51 in.) diameter seven-wire super strands having a nominal area of 100 mm² (0.155 in.²). The actual material properties, together with the other test data, are given in Table 1.

Test setup

To simulate actual behavior of slab-column connections, the slab was supported on eight vertical struts equipped with universal pinned joints at both ends of the strut. The pinned condition at the column bottom was formed by a column support resting on a ball bearing and held by two

NOTE: All dimensions are in millimeter (1 mm = 0.0394 in)

(a) Plan view

NOTE: All dimensions are in millimeter (1 mm = 0.0394 in)

(b) Elevation view – south

Fig. 3—Test setup.

steel arms in two perpendicular directions (Fig. 3). Below the column support, a 2000 kN (449.6 kip) hydraulic jack was provided to apply gravity loads to the slab. At one of the slab edges, a torsion-restraining frame was attached to prevent any in-plane rigid-body rotations during the testing. This frame is free to move in the east-west direction; hence, it will not affect the moment about the y -axis. The column

top was held by two 500 kN (112.4 kip) lateral hydraulic jacks in two perpendicular directions to apply lateral loads simulating earthquake loading. To apply axial force in the column, two prestressing rods equipped with a 1000 kN (224.8 kip) hydraulic jack each were installed passing through the column center. The details of the test setup are shown in Fig. 3.

Table 1—Test data

Specimen	h , mm	c_x , mm	c_y , mm	f'_c , MPa	d , mm	ρ_{tx} , %	ρ_{ty} , %	ρ_{bx} , %	ρ_{by} , %	f_y , MPa	d_{px} , mm	d_{py} , mm	ρ_{px} , %	ρ_{py} , %	f_{py} , MPa	f_{pu} , MPa	f_{se} , MPa	f_{se} , MPa	f_{pcx} , MPa	f_{pcy} , MPa	V_p , kN
PI-0	150	180	900	33.0	118	0.47	1.01	0.47	0.34	513	108	126	0.14	0.77	1779	1881	816	587	0.95	1.87	21.1
PI-1	150	180	900	36.1	118	0.47	1.01	0.47	0.34	513	108	126	0.14	0.77	1779	1881	782	533	0.91	1.70	19.7
PI-2	150	180	900	34.0	118	0.47	1.01	0.47	0.34	513	108	126	0.14	0.77	1779	1881	821	507	0.95	1.62	19.8

Notes: h is slab thickness; c_x , c_y are column widths in x - and y -directions, respectively; f'_c is compressive strength of concrete cylinder; d is effective depth of nonprestressed reinforcement; ρ_{tx} , ρ_{ty} are top nonprestressed reinforcement ratios within c plus $1.5h$ on each side of column in x - and y -directions, respectively; ρ_{bx} , ρ_{by} are bottom nonprestressed reinforcement ratios within c plus $1.5h$ on each side of column in x - and y -directions, respectively; f_y is yield strength of nonprestressed reinforcement; d_{px} , d_{py} are effective depths of tendons at column centerlines in x - and y -directions, respectively; ρ_{px} , ρ_{py} are prestressed reinforcement ratios within c plus $1.5h$ on each side of column in x - and y -directions, respectively; f_{py} is yield strength of prestressing steel; f_{pu} is tensile strength of prestressing steel; f_{se} , f_{se} are effective stresses in prestressing steel in x - and y -directions, respectively; f_{pcx} , f_{pcy} are compressive stresses in concrete due to all effective prestress forces at centroid of section in x - and y -directions, respectively; and V_p is vertical component of all effective prestress forces crossing critical section; 1 mm = 0.0394 in.; 1 MPa = 145 psi; 1 kN = 0.225 kip.

Fig. 4—Cyclic displacement history.

Test procedures

First, the column axial load was applied by stressing the two prestressing rods (Fig. 3) to 500 kN (112.4 kip) each, giving a total axial load of 1000 kN (224.8 kip), or 19% of the axial capacity of the column. This load was maintained throughout the test. Second, the slab tendons were stressed to the desired forces. Third, gravity loads in the slab were simulated by jacking up the bottom jack as well as by the slab self-weight. The gravity loads were introduced in a load-controlled mode up to the desired level. For Specimen PI-0, the gravity loads were applied until the specimen failed. For Specimens PI-1 and PI-2, the gravity load was applied up to a level that represents 100% of the dead load plus 25% of the live load in the model structure [$\cong 167$ kN (37.5 kip)]. This load was kept constant during the application of the cyclic lateral displacements. Fourth, after the application of the gravity loading, cyclic lateral displacements were applied at the column top by means of two lateral jacks. These lateral displacements were introduced in a displacement-controlled mode and followed the displacement history shown in Fig. 4. Specimen PI-1 was subjected to unidirectional cyclic displacements along (or parallel to) the strong column direction (y -direction), whereas Specimen PI-2 was subjected to bidirectional cyclic displacements according to the pattern shown in Fig. 5. Finally, the test was terminated when the strength of the specimen had dropped to approximately 70% of the ultimate strength.

Fig. 5—Bidirectional displacement pattern.

EXPERIMENTAL RESULTS AND DISCUSSION

All the specimens failed in punching shear, indicated by the column punching through the slab. Table 2 presents the experimental results for each specimen in terms of failure loads and drift capacities.

Bidirectional effect

The experiment under cyclic loading was done in a displacement-controlled mode so the M_{uy} and M_{ux} at the corner points of the diagram in Fig. 5 may not be the maximum value. Due to bidirectional effects, the moments at the corner points are lower than those about the principal axes. Referring to Fig. 5, the column top was first displaced to the east (Point 1), which introduces M_{uy} . Subsequently, when the column top was displaced to the north (Point 2), the value of M_{uy} decreased due to bidirectional effects. When the column top was displaced back to the west (Point 3), the value of M_{ux} was also reduced. Hence, the maximum unbalanced moments about each principal axis were obtained at Points 1, 5, 9, and 13. The M_{uy} and M_{ux} measured at the corner points (6 or 14) were 69.4 and 143.2 kN-m (51.2 and 105.6 kip-ft), respectively. The corresponding ultimate drift ratios (DR_{ux} and DR_{uy}) were 1.49% and 1.47%, respectively. The M_{ux} of Specimen PI-2 at corner points should not be compared with M_{ux} of Specimen PI-1 due to the presence of M_{uy} in Specimen PI-2. Note that the principal moment at the corner points should be $\sqrt{(69.4^2 + 143.2^2)} = 159.1$ kN-m (117.3 kip-ft). This value is lower than M_{ux} of 162.1 kNm (119.6 kip-ft), indicating that the failure did not necessarily occur at the corner points.

Fig. 6—Vertical column displacement versus drift ratio.

Vertical column displacements

The values of vertical column displacements recorded during the test are plotted against the drift ratios in Fig. 6. Note that the vertical column displacements were taken at the end of the first cycle of each target drift ratio. It was shown that the vertical column displacements increased as the drift ratio increased, even though the gravity load was kept constant during the application of the cyclic lateral displacements. This occurred due to the deterioration of the connection stiffness caused by cyclic lateral displacements. Furthermore, for the same drift ratio, Specimen PI-2 had a higher vertical column displacement compared to Specimen PI-1. This indicated that bidirectional loading caused more deterioration in stiffness than unidirectional loading.

Tendon forces

The average tendon forces for Specimen PI-0 were plotted against gravity load in Fig. 7. For Specimens PI-1 and PI-2, the average tendon forces were plotted against drift ratio (Fig. 8). It was shown that the tendon forces started to increase at a gravity load of approximately 150 kN (33.7 kip)—the load where the slab started to crack. As the gravity load and drift ratio increased further, the tendon forces also increased. At the ultimate limit state, the tendon forces of Specimen PI-0 were increased by 42.7 and 29.1 kN (9.6 and 6.5 kip) for x - and y -tendons, respectively. The increments of tendon forces in Specimens PI-1 and PI-2 were smaller than those in Specimen PI-0. The tendon forces of Specimen PI-1 were increased by 10.2 and 11.5 kN (2.3 and 2.6 kip) for x - and y -tendons, respectively. For Specimen PI-2, the tendon forces were increased by 10.6 and 13.4 kN (2.4 and 3.0 kip) for x - and y -tendons, respectively. Hence, at the ultimate limit state, specimens under pure shearing force have higher increments of tendon forces than specimens subjected to unbalanced moment transfer. Note that none of the tendons reached yield.

The increments of tendon forces at the ultimate limit state according to Eq. (18-3) of ACI 318-11 were calculated to be approximately 15.1 and 8.5 kN (3.4 and 1.9 kip) for x - and y -tendons, respectively. These values are close to the values recorded in Specimens PI-1 and PI-2, but differ significantly from the values obtained in Specimen PI-0. Note that Eq. (18-3) of ACI 318-11 was derived for tendons having an

Table 2—Test results

Specimen	V_u , kN	M_{uy} , kN-m	M_{ux} , kN-m	DR_{ux} , %	DR_{uy} , %
PI-0	511.8	—	—	—	—
PI-1	164.0	—	185.5	—	2.50
PI-2	170.6	79.8	162.1	1.49	1.52

Notes: V_u is shear force at failure; M_{uy} , M_{ux} are ultimate unbalanced moments acting at centroid of column section about y - and x -axes, respectively; DR_{ux} , DR_{uy} are ultimate drift ratios at peak unbalanced moment in x - and y -directions, respectively; 1 kN = 0.225 kip; 1 kN-m = 0.737 kip-ft.

Fig. 7—Average tendon forces versus gravity load.

f_{se} value greater than $0.5f_{pu}$, whereas the values of f_{se} in this experiment are less than $0.5f_{pu}$. In addition, the tendons in these tests are very short compared with those in actual structures. Hence, Eq. (18-3) of ACI 318-11 is good for normal tendon length, but can be less accurate for short tendons.

Hysteretic response

The overall behavior of slab-column connections under cyclic lateral loading can be shown in hysteretic curves of unbalanced moments versus drift ratios. Figures 9 and 10 present the hysteretic curves of unbalanced moments versus drift ratios for Specimens PI-1 and PI-2. The unbalanced moment at a particular drift ratio was obtained by multiplying the applied lateral load with the effective column height. The effective column height is defined as the distance between the lateral jacks at column top and the steel arms at column bottom. The drift ratio was calculated as the ratio of the imposed lateral displacement at the column top to the effective column height. For interior connections, the positive and negative unbalanced moments about an axis for the same drift ratio are expected to be approximately the same. However, due to the unsymmetrical formation of cracks and different distributions of prestress forces, these unbalanced moments are not exactly equal. For analysis purposes, the values of unbalanced moments for the same drift ratio will be taken as the average of the positive and negative values.

Effect of prestressing on shear strength

To investigate the effect of prestressing on shear strength, 47 slab-column connections were collected from the liter-

Fig. 8—Average tendon forces versus drift ratio.

ature (Silva et al. 2007; Lee 2004; Oliveira et al. 2004; Teng et al. 2004), including one specimen from the present study. The data collected consist of 41 connections having circular/square columns (Silva et al. 2007) and six connections having rectangular columns with a column aspect ratio of 5 (Lee 2004; Oliveira et al. 2004; Teng et al. 2004). All the connections that have rectangular columns were reinforced concrete specimens, except one specimen from the present study (PI-0), which is a post-tensioned specimen. Figure 11 shows the plot of the normalized shear strength against the normalized compressive stress in concrete due to prestressing. The shear strength was calculated at a critical section located at a distance $d/2$ from the face of column, in the shape of a closed rectangle. To minimize the effects of different concrete strengths f'_c , nonprestressed reinforcement ratios ρ , and slab effective depths d , the shear strength was normalized against $(f'_c \cdot 100\rho)^{1/3}(1 + d/1000)^{-1/2}$. The selection of $(f'_c \cdot 100\rho)^{1/3}(1 + d/1000)^{-1/2}$ as the normalizing quantity was based on the consideration that the shear strength of slabs is a function of $(f'_c \cdot 100\rho)^{1/3}(1 + d/1000)^{-1/2}$ (Teng et al. 2004). The tension reinforcement ratio ρ was calculated over the column width c plus $1.5h$ on each side of the column. The size effect factor $(1 + d/1000)^{-1/2}$ was based on fracture mechanics as proposed by Bazant and Cao (1987). It can be seen from Fig. 11 that connections having rectangular columns have lower shear strength compared with connections having circular/square columns. For $f_{pc} = 0$ (reinforced concrete slabs), the shear strength of slabs having rectangular columns with a column aspect ratio of 5 is only approximately 75% than that of slabs having circular/square columns. As the compressive stress in concrete due to prestressing increases, the shear strength also increases. However, the increase in shear strength due to prestressing for connections having rectangular columns with a column aspect ratio of 5 tends to be lower (only 22%) than those having circular/square columns. It may also be influenced by the tendon arrangements (uniform or banded).

Unbalanced moment capacity and drift capacity

The envelopes of the average unbalanced moments versus drift ratios are depicted in Fig. 12. It is shown that a connection with a rectangular column can carry higher unbalanced moment about the strong column axis (M_x) than about the

Fig. 9—Hysteretic curve of unbalanced moment versus drift ratio (PI-1).

weak column axis (M_y). At the ultimate limit state, the unbalanced moment about the strong column axis (M_x) was approximately twice that about the weak column axis (M_y) (refer to Specimen PI-2). In addition, by comparing the values of unbalanced moment about the strong column axis (M_x), one can see that the specimen under unidirectional loading (PI-1) has higher unbalanced moment capacity compared to the specimen under bidirectional loading (PI-2). Hence, bidirectional loading reduces the connection strength considerably. Figure 12 also shows that Specimen PI-1 failed at a drift ratio of 2.50%, whereas Specimen PI-2 failed at a drift ratio of 1.50%. Both specimens have the same value of gravity shear ratio $V_u/\phi V_c$. Thus, the specimen subjected to unidirectional loading has higher drift capacity compared with the specimen subjected to bidirectional loading.

To study the effect of prestressing on the unbalanced moment capacity, Specimen PI-2 will be compared with Specimen YL-H2 tested by Tan and Teng (2005). Both specimens are almost identical in every aspect, except the nonprestressed reinforcement ratio and the presence of prestressing. The envelopes of the hysteretic curves of Specimen YL-H2 are plotted in Fig. 12. However, the unbalanced moment capacity cannot be compared directly, as the flexural capacities of the two specimens are different. To minimize this effect, the unbalanced moment capacities M_u are normalized against the flexural capacity based on the yield-line theory M_{yl} (Gesund and Goli 1979). The ratios of the unbalanced moment capacity to the flexural capacity (M_u/M_{yl}) are also given in Fig. 12. For moments about the strong column axis, the ratios M_u/M_{yl} for Specimens PI-2 and YL-H2 are 0.28 and 0.22, respectively. This indicates that prestressing increases the unbalanced moment capacity. For moments about the weak column axis, the respective M_u/M_{yl} for Specimens PI-2 and YL-H2 are 0.37 and 0.50. This indicates that prestressing does not increase or reduce the unbalanced moment capacity. More data seems to be needed to draw a more definitive conclusion. In the tests by Gayed and Ghali (2006), the ratios of prestressed to non-prestressed reinforcement were changed but their flexural strengths were kept the same. Their post-tensioned connections were reinforced with the same amount of stud shear reinforcement. They concluded that prestressing increases the unbalanced moment strength.

(a) In weak column direction (x-dir)

(b) In strong column direction (y-dir)

Fig. 10—Hysteretic curve of unbalanced moment versus drift ratio (PI-2).

Connection ductility

Ductility of slab-column connections (μ) is usually defined as the ratio between the ultimate drift ratio and yield drift ratio ($\mu = DR_u/DR_y$). However, the connection ductility cannot be defined uniquely, as there is no distinct yield drift ratio on the unbalanced moment-drift ratio envelope curve due to the gradual spreading of yield across the slab transverse width. Pan and Moehle (1989) proposed an arbitrary procedure to determine a representative yield drift ratio. In their method, the envelope of unbalanced moment to drift ratio curve is idealized as a bilinear elastoplastic behavior. The elastic portion intersects the actual curve at a value of unbalanced moment equal to two-thirds of the maximum unbalanced moment. The yield drift ratio is then taken as the intersection between the elastic and plastic portion, whereas the ultimate drift ratio is taken at the maximum unbalanced moment. Megally and Ghali (2000) used a similar procedure as Pan and Moehle (1989) used to determine the connection ductility; however, the ultimate drift ratio is taken at 80% of the maximum unbalanced moment instead of at the maximum unbalanced moment. For comparison purposes, both values of connection ductility (μ and μ_{80}) are presented in Table 3.

It can be observed that bidirectional cyclic lateral loading significantly reduces the connection ductility. Comparing

Fig. 11—Normalized shear strength versus normalized precompression stress.

Fig. 12—Envelope of average unbalanced moment versus drift ratio.

Specimens PI-1 and PI-2, the connection ductility in the y-direction (μ_{80y}) is reduced from 2.82 to 1.75 due to bidirectional cyclic lateral loading. The column rectangularity does not seem to have any significant effect on the connection ductility, as Specimen PI-2 produces about the same values of connection ductility in x- and y-direction. This finding was also obtained in the tests by Tan and Teng (2005). The reason is probably due to the fact that once the connection fails in one direction, the strength in another direction dropped or was even lost. Thus, the values of connection ductility in the two orthogonal directions become similar. If unidirectional cyclic lateral loading was applied independently in two orthogonal directions, the connection ductility in x- and y-directions would be significantly different, as shown in the tests by Anggadajaja and Teng (2008). The connection ductility in the x-direction (weak column direction) is expected to be higher than that in y-direction (strong column direction).

To observe the effect of prestressing on the connection ductility, Specimen PI-2 will be compared with Specimen YL-H2 tested by Tan and Teng (2005). The values of the connections ductility of Specimen YL-H2 are also given in Table 3. The data show that prestressing may increase or reduce the connection ductility. Thus, there is no clear effect of prestressing on the connection ductility. Gayed and

Table 3—Connection ductility

Specimen	DR_{yx} , %	DR_{yy} , %	DR_{ux} , %	DR_{uy} , %	DR_{u80x} , %	DR_{u80y} , %	μ_x	μ_y	μ_{80x}	μ_{80y}
PI-0	—	—	—	—	—	—	—	—	—	—
PI-1	—	1.15	—	2.50	—	3.24	—	2.17	—	2.82
PI-2	1.14	1.16	1.49	1.52	2.24	2.03	1.31	1.31	1.96	1.75
YL-H2 (Tan and Teng 2005)	1.16	0.90	1.96	1.45	1.97	1.95	1.69	1.61	1.70	2.17

Notes: DR_{yx} , DR_{yy} are yield drift ratios in x- and y-directions, respectively; DR_{ux} , DR_{uy} are ultimate drift ratios at peak unbalanced moment in x- and y-directions, respectively; DR_{u80x} , DR_{u80y} are ultimate drift ratios at 80% peak unbalanced moment in x- and y-directions, respectively; μ_x , μ_y are ductility ratios corresponding to DR_y in x- and y-directions, respectively; and μ_{80x} , μ_{80y} are ductility ratios corresponding to DR_{u80} in x- and y-directions, respectively.

Fig. 13—Stiffness parameter S versus drift ratio.

Ghali (2006) also found that the level of prestressing did not consistently affect the connection ductility.

Connection stiffness

As the connection is loaded cyclically beyond its elastic range, the stiffness deteriorates. To measure the rate of stiffness deterioration, stiffness parameter S is adopted in this paper. The stiffness parameter S is defined as the slope of the line joining peak-to-peak points of individual hysteresis curves. Figure 13 shows the plots of stiffness parameter S (taken at the first cycle of each target drift ratio) versus drift ratio. It was shown that the connection stiffness in the strong column direction (y-direction) is much higher than the stiffness in the weak column direction (x-direction). In addition, bidirectional cyclic lateral loading reduces the connection stiffness significantly.

To see the effect of prestressing on the connections stiffness, the stiffness parameter S of Specimen YL-H2 tested by Tan and Teng (2005) is also plotted in Fig. 13. It can be seen that the reinforced concrete specimen (YL-H2) has higher stiffness compared with the post-tensioned specimen (PI-2). The reason may be due to the presence of ducts that reduce the effective area of the concrete and consequently reduce the stiffness. In addition, stiffness degradation of post-tensioned flat plates subjected to cyclic reversed displacements depends on the amount and distribution of both the nonprestressed and prestressed reinforcements in the column vicinity and in the direction of the cyclic reversed displacements. The tests by Gayed and Ghali (2006) also showed that the presence of prestressing had no consistent trend in the connection stiffness. Thus, no definitive conclusions can be drawn.

Fig. 14—Ultimate drift ratio DR_u versus gravity shear ratio V_u/V_c .

ACI PROVISIONS ON PUNCHING SHEAR

ACI 318-11 defines the punching shear strength of reinforced concrete slabs without shear reinforcement as the smallest of Eq. (11-31) to (11-33). For post-tensioned slabs without shear reinforcement, the punching shear strength can be calculated from Eq. (11-34). ACI 318-11 requires that the use of Eq. (11-34) be restricted to the following conditions:

- No portion of the column cross section shall be closer to a discontinuous edge than $4h$;
- The value of $\sqrt{f'_c}$ shall not be taken greater than 70 psi (5.8 MPa); and
- The value of f_{pc} in each direction shall be between 125 and 500 psi (0.9 and 3.5 MPa).

If one of those conditions is not satisfied, the calculation of v_c shall be based on formula for reinforced concrete slabs (Eq. (11-31) to (11-33)). However, Foutch et al. (1990) and Silva et al. (2007) concluded that the limitations of concrete strength and compressive stress in ACI 318 are not necessary, as they do not improve the correlations with test results. Hence, in this paper, the strength predictions of post-tensioned slabs by ACI 318-11 were calculated using Eq. (11-34), ignoring the limitations of concrete strength and compressive stress. In addition, the value of λ is always taken to be unity, as this study is limited only to slab-column connections cast with normalweight concrete.

If there is a transfer of unbalanced moment M_u between the slab and column, ACI 318-11 presents a method known as the eccentric shear stress model to calculate the ultimate

Table 4—Analysis results

Specimen	v_c , MPa	v_u , MPa	v/v_c	M_{fy} , kN-m	$\gamma_{fy}M_{uy}$, kN-m	$\gamma_{fy}M_{uy}/M_{fy}$	M_{fx} , kN-m	$\gamma_{fx}M_{ux}$, kN-m	$\gamma_{fx}M_{ux}/M_{fx}$	Ratio	FM
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
PI-0	2.06	1.65	0.80	105.5	0.0	0.00	110.4	0.0	0.00	0.80	S
PI-1	2.10	1.86	0.88	105.5	0.0	0.00	109.3	83.1	0.76	0.88	S
PI-2	2.04	2.24	1.10	105.8	58.6	0.55	106.9	72.6	0.68	1.10	S

Notes: v_c is shear strength; v_u is ultimate shear stress; M_{fy} , M_{fx} are flexural capacities within c plus $1.5h$ on each side of column about y - and x -axes, respectively; γ_{fy} , γ_{fx} are fractions of unbalanced moments transferred by flexure about y - and x -axes, respectively; M_{uy} , M_{ux} are ultimate unbalanced moments acting at centroid of column section about y - and x -axes, respectively; Ratio is maximum value of Columns (4), (7), and (10); and FM is failure mode based on the ratio (S = shear, F = flexure); 1 MPa = 145 psi; 1 kN-m = 0.737 kip-ft.

shear stresses acting on the critical section. The model assumes that a portion of unbalanced moment ($\gamma_{fy}M_{uy}$) shall be transferred by flexure within an effective slab width (c plus $1.5h$ on each side of the column) and the remaining unbalanced moment ($\gamma_{fy}M_{uy}$) shall be transferred by eccentric shear about the centroid of the critical section. The fractions of the unbalanced moments transferred by flexure (γ_{fy}) and eccentric shear (γ_{vy}) are defined in Eq. (13-1) and Eq. (11-37) of ACI 318-11, respectively. The shear stress resulting from gravity forces is assumed to be uniform along the critical section, whereas the shear stress resulting from unbalanced moment transfer is assumed to vary linearly about the centroid of critical section. Thus, the maximum shear stress v_u due to combined shear and moment transfer can be calculated from

$$v_u = \frac{V_u}{b_o d} \pm \frac{\gamma_{vx} M_{ux} s_x}{J_{cx}} \pm \frac{\gamma_{vy} M_{uy} s_y}{J_{cy}} \quad (1)$$

The subscripts x and y represent the axes about which unbalanced moments are transferred. The definition of J_c is ambiguous, as polar moment of inertia is known, in mechanics, as the second moment of a plane area about a point, not about an axis (x or y), but the assumed shear critical section is a spatial nonplane surface. The ambiguity of the definition of J_c can be a hurdle in accounting for the unbalanced moment in punching shear design of connections having non-rectangular shapes. ACI 421.1R-08 (Joint ACI-ASCE Committee 421 2008) gives a general equation that applies to any shear critical section having a polygonal shape.

For connections of flat plates with rectangular columns, v_u is larger when b_1 exceeds b_2 (refer to Eq. (11-37) and (13-1) of ACI 318-11). High shear stress results on the b_2 sides of the shear critical section due to the transfer of unbalanced moment about an axis parallel to the short side of the column (for the specimens in Fig. 1 to 3, the term containing $\gamma_{vx} M_{ux}$ of Eq. (1)). Shear strength of a connection (v_n) is a function of v_c and v_s (if shear reinforcement is provided), while the shear stress is induced by the internal forces at the slab-column connection: V_u , M_{ux} , and M_{uy} .

When a flat plate structure is assumed not to contribute to the seismic-force-resisting system, the slab-column connections shall be designed to resist the gravity shear force V_u and the induced moment transferred under the design displacement. Alternatively, the design story drift ratio should not exceed the larger of 0.005 and $[0.035 - 0.05(V_u/\phi V_c)]$. The

gravity shear ratio $V_u/\phi V_c$ is defined as the ratio of the gravity shear force V_u to the shear capacity V_c , given by

$$V_c = v_c b_o d \quad (2)$$

where v_c is calculated using Eq. (11-31) to (11-34). In this paper, the strength reduction factor ϕ is taken equal to unity. If the design story drift ratio exceeds the limit given above, then slab shear reinforcement shall be provided. This requirement is based on a recommendation from ACI 352.1R-89 (Joint ACI-ASCE Committee 352 1989) that the gravity shear ratio should be kept below 0.4 to ensure some minimal ductility with the availability of approximately 1.5% drift capacity.

COMPARISON OF ACI 318-11 PREDICTIONS AND EXPERIMENTAL RESULTS

Strength predictions

Table 4 presents the analysis results calculated using the ACI eccentric shear stress model. The ratios of the experimental strength to the predicted value based on shear failure, flexural failure about the y -axis, and flexural failure about the x -axis are shown in Columns (4), (7), and (10), respectively. The strength ratio in Column (11) of Table 4 is the maximum of the three ratios—namely, v_u/v_c , $\gamma_{fy}M_{uy}/M_{fy}$, and $\gamma_{fx}M_{ux}/M_{fx}$. Column (12) of Table 4 gives the predicted failure mode (shear or flexure) based the governed values of Column (11). This analysis indicates that all the specimens failed in shear, which is in line with the experimental failure mode. In addition, the ACI 318-11 method is only conservative for Specimen PI-2 and unconservative for the other two specimens (PI-0 and PI-1). Hence, the ACI 318-11 method can be unconservative for post-tensioned slab-column connections having rectangular columns.

Proposed modification to β_p

The ACI 318-11 formula for shear capacity of prestressed slabs (Eq. (11-34)) does not take into account the effect of column rectangularity. Hence, it is likely to be the main reason why ACI 318-11 can be unsafe for post-tensioned connections having rectangular columns. A simple modification can be obtained by introducing the column rectangularity factor into the β_p -coefficient in the formula for prestressed slabs in the same way as the v_c formula for nonprestressed slabs. Hence, the shear strength formula for

prestressed slabs becomes (Eq. (11-34) of ACI 318-11 is rewritten for clarity)

$$v_c = \beta_p \lambda \sqrt{f'_c} + 0.3 f_{pc} + \frac{V_p}{b_o d} \quad (3)$$

where β_p is now defined as

$$\beta_p = \min \begin{cases} 3.5 \\ (\alpha_s d / b_o + 1.5) \text{ (psi)} \\ (1.5 + 4 / \beta) \end{cases} \quad (4)$$

or

$$\beta_p = \min \begin{cases} 0.29 \\ 0.083(\alpha_s d / b_o + 1.5) \text{ (MPa)} \\ 0.083(1.5 + 4 / \beta) \end{cases}$$

Note that the first two β_p were similar to Eq. (11-33) and (11-32), except that ACI reduces the shear strength by $0.5\sqrt{f'_c}$ psi ($0.06\sqrt{f'_c}$ MPa). It is proposed to add the third β_p , which is similar to Eq. (11-31), and reduce the strength by $0.5\sqrt{f'_c}$ psi ($0.06\sqrt{f'_c}$ MPa).

The shear strengths v_c calculated using the modified β_p -factor are 1.59, 1.60, and 1.56 MPa (230.5, 232, and 226.2 psi) for Specimens PI-0, PI-1, and PI-2, respectively. Consequently, the shear strength ratios v_u/v_c become 1.04, 1.16, and 1.43 for Specimens PI-0, PI-1, and PI-2, respectively. As this modification leads to a more conservative result and also represents the failure modes correctly, Eq. (4) is therefore proposed to include the column rectangularity into the ACI 318-11 equation for prestressed slabs (Eq. (11-34)).

Gravity-shear-ratio requirement

To check the ACI 318-11 requirement on the gravity shear ratio, 14 experimental data (including two data from the present study) were collected from the literature (Trongtham and Hawkins 1977; Dilger and Shatila 1989; Martinez-Cruzado et al. 1994; Han et al. 2006). All the specimens were interior post-tensioned slab-column connections without shear reinforcement and subjected to cyclic lateral loading. The term "interior" in this paper refers to connections with four sides of critical section. Hence, one specimen of edge connection with overhanging slab (Dilger and Shatila 1989) was also included in this study. The specimens tested by the authors were connections having rectangular columns with a column aspect ratio of 5, whereas the rest were connections having square columns. Figure 14 shows the plots of drift capacities at failure (DR_u) against gravity shear ratios ($V_u/\phi V_c$), with ϕ taken equal to 1.0. For the authors' specimens, the gravity shear ratios were calculated using both the original and the modified β_p -factor. It can be seen that Specimen PI-2 still falls below the bilinear line according to the ACI 318-11 requirement even though the modified β_p -factor was used. This indicates that the ACI 318-11 requirement on gravity-shear-ratio can still be unsafe for a post-tensioned slab-column connection having rectangular columns with a

column aspect ratio of 5 and subjected to bidirectional cyclic lateral loading. More data are certainly needed.

CONCLUSIONS

Based on the results of this study, the following conclusions can be made:

1. The strength and stiffness of slab-column connections having rectangular columns with a column aspect ratio of 5 are much higher along the strong-column direction than those along the weak-column direction.
2. Cyclic lateral displacements cause deterioration of the connection stiffness, and the effect of bidirectional loading is more severe than unidirectional loading. It was shown that bidirectional cyclic lateral loading significantly reduced the unbalanced moment capacity, drift capacity, ductility, and stiffness of slab-column connections.
3. Prestressing increases the shear strength of slab-column connections. The increase in shear strength due to prestressing for connections having rectangular columns tend to be lower compared to those with circular/square columns.
4. The ACI 318-11 equation on shear strength can be unconservative for post-tensioned connections having rectangular columns. Hence, a modification on the shear strength formula is proposed to make the strength prediction more conservative.
5. The ACI 318-11 requirement on gravity shear ratio can be unsafe for post-tensioned slab-column connections having rectangular columns with a column aspect ratio of 5 and subjected to bidirectional cyclic lateral loading.

AUTHOR BIOS

Aris Himawan is a Structural Engineer at Arup Singapore Pte Ltd. He received his BEng from Parahyangan Catholic University, Bandung, Indonesia, and his MSc and PhD from Nanyang Technological University, Singapore. His research interests include behavior of reinforced and prestressed concrete flat plate structures.

ACI Member Susanto Teng is an Associate Professor at the School of Civil and Environmental Engineering, Nanyang Technological University. He is a member of ACI Committee 435, Deflection of Concrete Building Structures; and Joint ACI-ASCE Committees 421, Design of Reinforced Concrete Slabs; and 445, Shear and Torsion. His research interests include behavior of slab-column connections, flexural and shear behavior of irregular flat plate slabs, shear in beams, and finite element modeling.

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NOTATION

b_1, b_2	= widths of shear critical section measured parallel and perpendicular to moment considered, respectively
b_o	= perimeter length of shear critical section
c	= column width
DR_u	= ultimate drift ratio at peak unbalanced moment
DR_{u80}	= ultimate drift ratio at 80% of peak unbalanced moment
DR_y	= yield drift ratio
d	= effective depth of nonprestressed reinforcements running in two orthogonal directions
d_p	= effective depth of tendon at column centerline
f'_c	= compressive strength of concrete cylinder
f'_{cu}	= compressive strength of concrete cube
f_{pc}	= compressive stress in concrete due to all effective prestress forces at centroid of section
f_{pu}	= tensile strength of prestressing steel

f_{py}	=	yield strength of prestressing steel
f_{se}	=	effective stress in prestressing steel
f_y	=	yield strength of nonprestressed reinforcement
h	=	slab thickness
J_c	=	property of assumed critical section analogous to polar moment of inertia
M_f	=	slab flexural strength within c plus $1.5h$ on each side of column
M_u	=	ultimate unbalanced moment acting at centroid of column section
M_{yl}	=	slab flexural strength based on yield-line theory
S	=	stiffness parameter
s	=	distance from centroidal axis to extreme fibers of critical section
V	=	shear capacity provided by concrete
V_p	=	vertical component of all effective prestress forces crossing critical section
V_u	=	ultimate shear force
v_c	=	shear strength provided by concrete
v_n	=	shear strength
v_s	=	shear strength provided by shear reinforcement
v_u	=	ultimate shear stress
α_s	=	constant used to compute v_c in slabs
β	=	ratio of long to short side of column
β_p	=	factor used to compute v_c in prestressed slabs
ϕ	=	strength reduction factor
γ_f	=	fraction of unbalanced moment transferred by flexure
γ_v	=	fraction of unbalanced moment transferred by eccentric shear
λ	=	modification factor reflecting reduced mechanical properties of lightweight concrete
μ	=	connection ductility corresponding to DR_u
μ_{80}	=	connection ductility corresponding to DR_{u80}
ρ_b	=	bottom nonprestressed reinforcement ratio within c plus $1.5h$ on each side of column
ρ_p	=	prestressed reinforcement ratios within c plus $1.5h$ on each side of column
ρ_t	=	top nonprestressed reinforcement ratio within c plus $1.5h$ on each side of column

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NOTES: