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Punching shear often controls the required slab thickness or column size of flat-plate slabs. This paper presents the results of an experimental investigation of the punching shear strength of four post-tensioned (PT) concrete slabs. Two tests were performed on PT concrete slabs with L-shaped columns and the other two were performed with square columns. The punching shear results are compared with ACI 318 equations and those proposed by other investigators.

KEYWORDS

L-shaped columns; post-tensioned slab; punching shear.

INTRODUCTION

Shear strength of flat slabs in the vicinity of columns or concentrated loads is often controlled by the punching shear strength. Shear considerations can therefore be the controlling factor in determining the required slab thickness or column size, especially of post-tensioned (PT) flat plates. Tests on flat plates[1] have shown that if shear failures are prevented, almost complete redistribution of bending moments can be achieved prior to failure. Punching shear occurs by cracking along the surface of a truncated cone or pyramid around the column. For a reinforced concrete flat slab, the inclined surface of the truncated pyramid is typically assumed to make a 45-degree angle with the top surface of the slab. For design purposes,[2] the critical section is assumed at $d/2$ from the column face, where d is the distance from the extreme compression fiber to the centroid of longitudinal tension reinforcement, taken not less than 80% of the overall slab thickness for PT slabs. Although the same critical sections are used for PT slabs, tests[3-6] performed on prestressed flat slabs have indicated that the critical shear area can be larger than a section assumed to be bounded by $d/2$ from the column face. Variables that affect the punching shear strength of a PT slab include the concrete compressive strength, the effective depth, the dimensions of the critical section, and the effective force in the prestressing tendons.

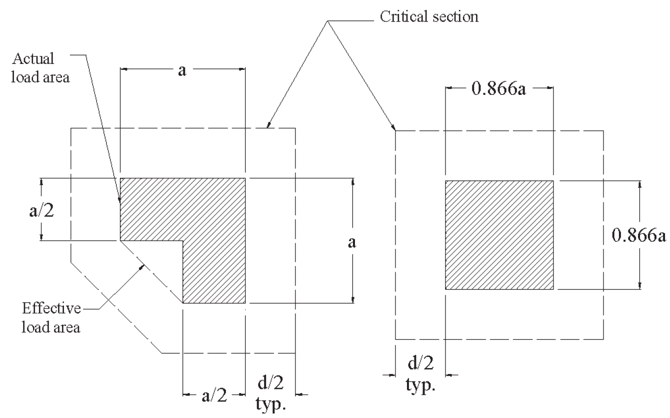

Fig. 1—Critical shear perimeter for nonrectangular and square loaded areas.

Figure 1 provides a comparison of the critical shear perimeters between an L-shaped column and square columns of the same cross-sectional area, with the critical shear perimeter taken at a distance $d/2$ from the effective loaded area. The larger effective area of the L-shaped column results in a higher punching strength. The better shear performance and the ability to integrate the smaller column dimensions within perpendicular partition walls have increased the use of nonrectangular columns in PT slabs. More experimental results are needed, however, to better understand the behavior of PT slabs with nonrectangular columns.

The ACI 318 equation for punching shear strength is based on the analysis of the state of stress in prestressed concrete beams,[7] as shown in the following.

The shear stress v_{cw} and the compressive stress due to

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the prestressing f_{pc} produce a principal tensile stress given by

$$f'_t = -\frac{f_{pc}}{2} + \sqrt{\left(\frac{f_{pc}}{2}\right)^2 + v_{cw}^2} \qquad (1)$$

Solving for v_{cw} in Eq. (1) gives

$$v_{cw} = f'_t \sqrt{1 + \frac{f_{pc}}{f'_t}} \qquad (2)$$

Letting the tensile strength $f'_t = 3.5\sqrt{f'_c}$ $(0.29\sqrt{f'_c})$, the critical shear stress in Eq. (2) becomes

$$v_{cw} = 3.5\sqrt{f'_c}\sqrt{1 + \frac{f_{pc}}{3.5\sqrt{f'_c}}} \qquad (3)^*$$

which may be approximated as

$$v_{cw} = 3.5\sqrt{f'_c} + 0.3 f_{pc} \qquad (4)^*$$

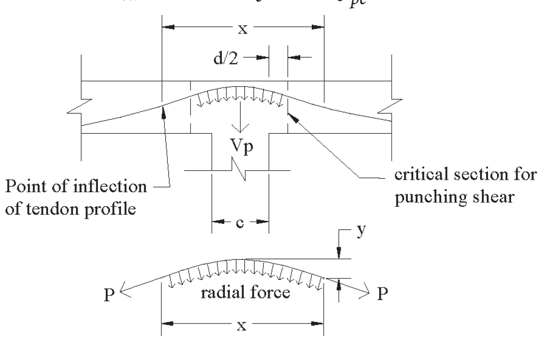

Fig. 2—Tendon profile in column region.

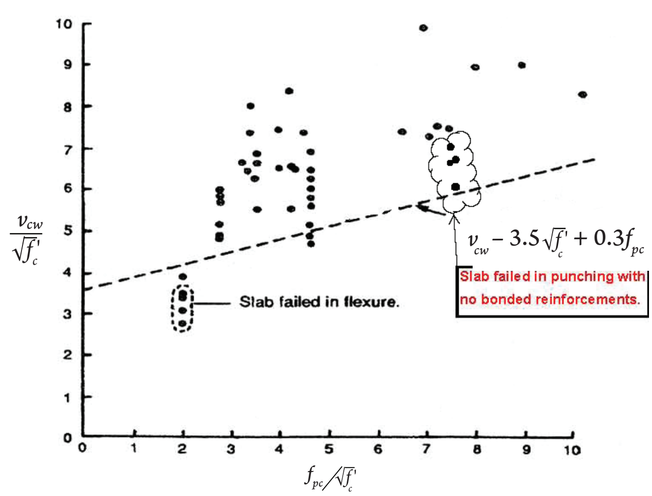

Fig. 3—Shear test data for reinforced concrete slabs plotted with ACI 318 equation.[8]

Equation (4) was modified by ACI 318[8] to obtain the following expression for the punching shear capacity of prestressed concrete slabs

$$V_c = (\beta_\varphi\sqrt{f'_c} + 0.3f_{pc})b_o d + V_p \qquad (5)^*$$

where β_φ is the smaller of 3.5 or $(\alpha_s d/b_o + 1.5)$ with α_s = 40 for an interior slab-column connection, f'_c is the concrete compressive strength, d is the effective depth of the PT slab, f_{pc} is the average compressive stress due to the prestressing force in both directions "after allowance for all losses," and not to exceed 500 psi (3.4 MPa) and b_o is the perimeter of the critical section. The variable V_p is the vertical component of all the prestressing forces crossing the critical section, which can be expressed with reference to (Fig. 2) as follows

$$V_p = \Sigma \frac{8Py}{x^2}(c + d) \qquad (6)$$

Results obtained from punching shear tests of a number of prestressed concrete slabs with rectangular columns have indicated that Eq. (4) provides a conservative estimate for the punching shear strength (Fig. 3).[9]

The current investigation was carried out to verify the ACI 318 predictions[8] for the punching shear capacity of PT slabs with nonrectangular columns as relatively little data exist on the behavior of nonrectangular columns.

LITERATURE REVIEW

The shear strength of prestressed flat plates has been studied in a variety of test specimens by several investigators. The results of these previous experimental investigations can be summarized as follows.

Tests by Scordelis et al.[3]

In 1958, Scordelis et al.[3] tested fifteen 6 ft (1.8 m) square slabs to study the ultimate shear strength of lift slabs. Twelve of the slabs were prestressed with unbonded tendons. The specimens were loaded at the center of the specimen and were supported along all four edges. The major variables for the tests were concrete strength, average prestressing force, slab thickness, size of the steel lift-collar, and the amount of collar recess.

All of the slabs failed by the punching of the

*in.-lb units	SI units
$3.5\sqrt{f'_c}$	$0.29\sqrt{f'_c}$
$\beta\varphi\sqrt{f'_c}$	$0.083\beta\varphi\sqrt{f'_c}$

lift-collar through the concrete. The ultimate shear stress at a distance $d/2$ from the edge of the collar varied from $5.4\sqrt{f'_c}$ to $8.9\sqrt{f'_c}$ ($0.45\sqrt{f'_c}$ to $0.72\sqrt{f'_c}$).

The tests showed that the ultimate shear strength of the slabs increased with the average prestress. The following empirical equation was proposed by them for the ultimate shear strength of prestressed concrete slabs

$$V_c = (0.175 - 2.42 \times 10^{-5} f'_c + 2.0 \times 10^{-5} F_e/s_t) b_p d f'_c \quad (7)$$

where V_c is ultimate shear, in lb (kN); F_e is the effective prestress force per tendon, in lb (kN); s_t is the tendon spacing, in in. (mm); b_p is the perimeter of the lifting collar, column, or column capital, in in. (mm); d is the effective depth, in in. (mm); and f'_c is the concrete compressive strength, in psi (MPa).

Tests by Grow and Vanderbilt[4]

In 1967, Grow and Vanderbilt[4] reported results from tests conducted on 10 prestressed lightweight-aggregate concrete flat plates. All of the test specimens were 3 ft (900 mm) square, 3 in. (75 mm) thick, and had a column stub at the center. The effective prestress was the only variable and ranged from 6 to 656 psi (0.04 to 4.5 MPa). The ultimate shear strength at a distance of $d/2$ from the column face varied from $5.1\sqrt{f'_c}$ to $7.9\sqrt{f'_c}$ ($0.42\sqrt{f'_c}$ to $0.66\sqrt{f'_c}$). The following equation was proposed for the ultimate shear strength

$$V_c = (360 + 0.3 f_{pc}) b_p d \quad (8)$$

where V_c is the shear strength, in lb (kN); f_{pc} is the average effective prestress, in psi (MPa); b_p is the perimeter of the column, in in. (mm); and d is the effective depth, in in. (mm).

Tests by Gerber and Burns[5]

In 1967, Gerber and Burns[5] reported tests by the American BBR Research Association on 10 prestressed normalweight concrete column-slab specimens. All of the specimens, except one, contained unbonded tendons. Six of the specimens were built to simulate lift-slab construction. The specimens were 12 ft (3.6 m) square, 7 in. (175 mm) thick, with precast columns supported at the center. The average prestress was 250 psi (1.7 MPa) for all the specimens. The test variables were tendon spacing and the amount and distribution of bonded reinforcement. The shear strength at failure varied from $4.1\sqrt{f'_c}$ to $6.2\sqrt{f'_c}$ ($0.34\sqrt{f'_c}$ to $0.51\sqrt{f'_c}$) on a critical section at a distance of $d/2$ from the column face or collar. The tests showed that the bonded reinforcement located in the column area at the top of the slab resulted in shear strength increases up to 14%.

Tests by Smith and Burns[10]

In 1974, Smith and Burns[10] reported results from tests conducted on three PT flat plate specimens. The three specimens were designed following the "Tentative Recommendations for Prestressed Concrete Flat Plates" of Joint ACI-ASCE Committee 423.[9] The test structure represented the region around an interior column of multiple-panel slabs and consisted of a 9 ft (2.7 m) square, 2.75 in. (70 mm) thick slab with a single 8 x 8 in. (200 x 200 mm) column stub in the center. The tendon spacing in each of the specimens corresponded to a 70% distribution in the column strip and a 30% distribution in the middle strip. The concrete prestress force f_{pc} was 325 psi (2.24 MPa). The variable investigated was the amount of bonded reinforcement (0 to 0.24% of the cross-sectional area of the column strip). The specimens were supported at the column stub and loading was accomplished by producing a patch load measuring 4 ft (1.2 m) square about the column centerlines.

All of the specimens failed in a combination of flexure and shear, with the final failure mode being one of punching shear. Shear strengths for these tests varied from $4.45\sqrt{f'_c}$ to $5.16\sqrt{f'_c}$ ($0.37\sqrt{f'_c}$ to $0.43\sqrt{f'_c}$). The lowest strength was in the slab with no bonded reinforcement.

Tests by Hawkins and Trongtham[11]

In 1976, Hawkins and Trongtham[11] presented a report on the testing of five unbonded, PT flat plate specimens. Four of the specimens simulated interior slab-column connections and were 13 ft (4 m) long, 7 ft (2.1 m) wide, and 5.5 in. (140 mm) thick with a 14 in. (350 mm) square column in the center. The other specimen was 7 x 7 ft (2.1 x 2.1 m), 5.5 in. (140 mm) thick with a 14 in. (350 mm) square at the edge and represented a typical exterior slab-column connection. In each specimen, the column extended 4 ft (1.2 m) above and 4 ft (1.2 m) below the slab and was prestressed to 50 kips (220 kN) to simulate an axial load.

Tendon layout was one variable studied in the interior slab-column specimens. Combinations of bundled and distributed tendon arrangements were examined in these tests. In the exterior column-slab connection specimen, the tendons were effectively distributed through the slab, with two tendons through the slab in either direction.

The concrete prestress f_{pc} was 150 psi (1.03 MPa) in each specimen. Bonded reinforcement in the column area was 10% less than that required by the January 1976 Proposed Revision to ACI 318[9] in the interior column specimens. The exterior column regions contained one-half the bonded column strip reinforcement of the interior column regions. Based on the results of their testing, Hawkins and Trongtham[11] recommended the ultimate shear strength be given by $v_c = ((3.5\sqrt{f'_c} + 0.3f_{pc}) + v_p)$ for unbonded PT prestressed concrete slab-column connections, which transfer moment $3.5\sqrt{f'_c}$ $(0.29\sqrt{f'_c})$. Other recommendations were also made for the distributions of bonded and unbonded reinforcement at the slab-column connections.

Tests by Kosut and Burns[12]

In 1985, Kosut and Burns[12] reported the shear strength and behavior of the slab-column connections of a four-panel 20 ft 8 in. x 20 ft 8 in. x 2.75 in. (6.3 m x 6.3 m x 70 mm) concrete slab with nine columns at 10 ft (3 m) centers. Four of the columns were 7 x 7 in. (175 x 175 mm) and five of the columns were 8 x 8 in. (200 x 200 mm). A banded tendon distribution was used with an effective prestress of 180 psi (1.24 MPa). The slab underwent two test series. The first series involved nine tests and was conducted to determine the slab's flexural behavior and strength. The second test series involved four tests conducted to determine the shear strength and behavior of several of the slab-column connections. The major variables for the shear tests were the column size, the inclusion of stirrups at some of the exterior column-connections tested, and the column location (corner, edge, or interior column). It was found that the shear strength of each of the slab-column connections tested was greater than that predicted by Eq. (5).[8] In addition, the tests showed that the stirrups at the exterior slab-column connections did not increase the ultimate shear stress. It was also found that the moment-transfer contribution to the total shear stress can be quite significant.

Tests by Burns and Hemakom[6]

In 1985, investigation was undertaken to observe the strength and behavior of a one-half scale, nine panel flat plate with banded arrangement of unbonded tendons. The test slab had three 10 ft (3 m) spans in each direction and 2.5 ft (750 mm) overhangs on two edges with the nominal thickness measured at 2.75 in. (70 mm). The slab was designed with a low P/A stress level of 135 psi (0.93 MPa). The overall performance of the test slab at service load level (50 psf [2.4 kN/m^2]) was quite satisfactory. The slab behaved elastically and deflection was fully recovered upon releasing the applied load. The slab before failure was very ductile, with large deflections observed in all tests to failure. The failure load was observed at 160 psf (7.7 kN/m^2), which was in excess of the designed factored load. The punching shear failure was secondary to the flexural failure; thus, the punching shear load was equal to the flexural failure load. The minimum bonded reinforcement of 0.15% of the column strip area provided very good crack control up to the failure load.

RESEARCH SIGNIFICANCE

The primary goal of this research was to conduct an experimental investigation of the punching shear behavior of PT slabs with L-shaped columns. The slabs were designed to ensure punching shear failure and the tests were conducted by applying a central concentrated load on the PT slabs. The tests were carried out with a distributed tendon layout in both directions, which is different from previous tests, thus adding to the database of available test results. The tests were also conducted without bonded steel reinforcement to obtain a lower bound value of the punching shear strength.

EXPERIMENTAL INVESTIGATION

Four PT concrete slabs with the same dimensions and post-tensioning were tested in the current study. Two of the tests were conducted with L-shaped columns designated as PT-a and PT-b and the other two tests were conducted with square columns designated as PT-c and PT-d. The test specimens were 6 ft x 6 ft x 4 in. (1.8 m x 1.8 m x 100 mm). The dimensions of the square column and the L-shaped column are shown in Fig. 4. The punching shear failure of the PT slabs was ensured by the following approach: the dimensions and thickness of the slab were chosen to yield a punching strength of approximately 40 kips (180 kN), (two-thirds of the capacity of the loading jack used for the tests) using Eq. (5).[8] The slab was then analyzed using a post-tensioning design program[13] considering the connections between the columns and the PT slab as pinned connections. The stress contours obtained by applying a load of 60 kips (267 kN), (capacity of the jack) as a patch load is shown in Fig. 5. The results indicate that the maximum compressive stress in the concrete at the critical sections is below 75% of the concrete compressive strength, which would ensure a punching shear failure mode of the PT slab. The numbers and the profiles of the prestressing cables were then

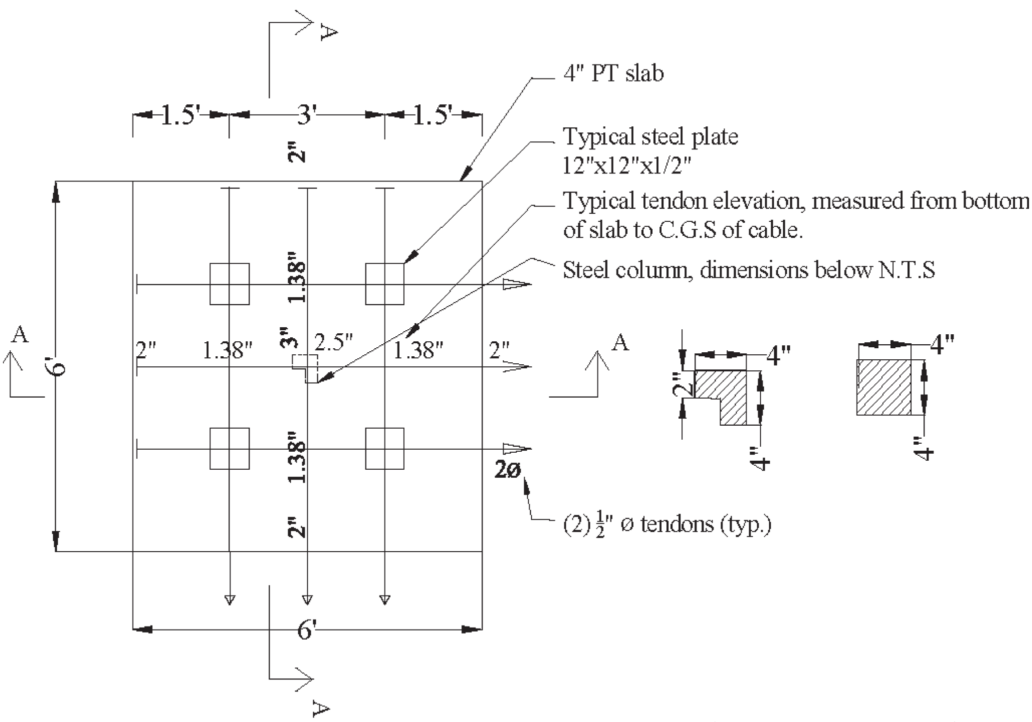

Fig. 4—Post-tensioning tendon layout in post-tensioned slabs. (Note: 1 in. = 25.4 mm.)

determined using a post-tensioning design program[13] and are shown in Fig. 4.

Materials

The concrete used for the slabs had a 28-day design compressive strength of 3500 psi (24.1 MPa) and was supplied by a local ready mixed concrete plant. The coarse aggregate used was No. 16 (1.18 mm) crushed limestone and the water-cement ratio (w/c) was 0.42. PT-a and PT-b were cast from one batch and PT-c and PT-d were cast later from a different batch of concrete. Five standard cylinders 6 in. (150 mm) in diameter by 12 in. (300 mm) in height were cast from each batch and kept in the same environment as the slab. For PT-a and PT-b, the compressive strength at the time of testing at 28 days was 4250 psi (29.3 MPa). For PT-c and PT-d, the compressive strength at the time of testing at 28 days was 4350 psi (30 MPa).

The prestressing strands used were 0.5 in. (12.7 mm) diameter seven-wire strands conforming to ASTM A416, with a specified ultimate strength of 270 ksi (1860 MPa) and an average modulus of elasticity of 28,600 ksi (197,000 MPa). The tendons were protected with a plastic sheathing to prevent the cable from bonding with the concrete and to reduce friction at the time of stressing.

Fig. 5—Principal compressive stress distribution for PT-a (applied load P *= 60 kips [267 kN]).*

Fabrication of specimens

The slabs were cast on the floor over plastic sheets. Prior to casting the slab, the plastic sheets were oiled for ease of removal of the specimens. Chairs were used to ensure that the desired tendon profile was attained. Ready mixed concrete was delivered and pumped into the molds. The concrete was consolidated by vibration

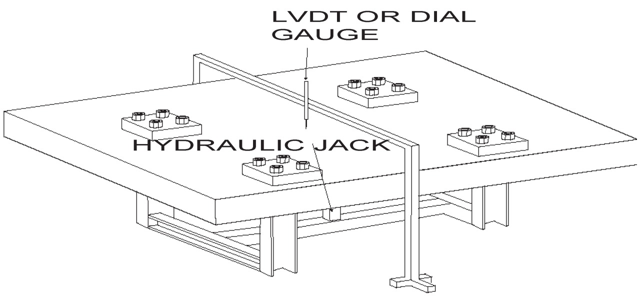

Fig. 6(a)—Schematic diagram of punching shear test setup.

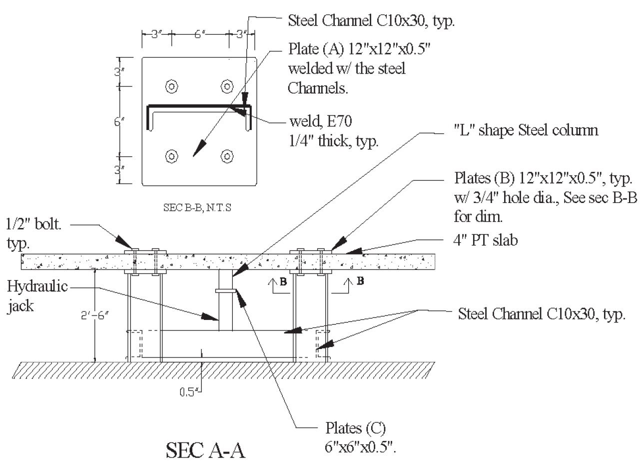

Fig. 6(b)—Steel reaction frame details. (Note: 1 in. = 25.4 mm.)

Fig. 6(c)—Photograph of post-tensioned slab before test.

and the final slab finish was achieved using steel trowels and wooden floats. A plastic cover was placed over the slab for a 15-day curing period.

Test setup and instrumentation

The specimens were tested in an elevated position using a steel test frame. The application of an upward load allowed for the observation of the punching shear failure on the top surface of the slab. The reaction frame was designed and fabricated of structural steel as shown schematically in Fig. 6(a). The central loading was applied upwards and the concrete slab was held down at four locations, which were 3 ft (900 mm) apart. The slab also had a 1.5 ft (450 mm) overhang on each side. Additional details of the anchorage of the slab to the structural steel frame are shown in Fig. 6(b). A photograph of the PT slab mounted on the testing frame is shown in Fig. 6(c). The tendons were stressed up to 33 kips (147 kN) after the slab was positioned on the test frame. The prestress loss was quite significant due to the seating losses that occurred with the ordinary wedges and due to the short length of the strands. The lowest effective tendon force after the seating losses was 16.5 kips (73.4 kN), as shown in Table 1.

The loading was accomplished by using a hydraulic jack. The hydraulic jack was previously calibrated to obtain the applied load. A dial gauge was mounted at the center of the slab to measure the displacement of the slab. Load cells were also placed between the edge of the slab and the anchoring mechanism of the prestressing tendons to measure the force in the four central tendons, as indicated in Fig. 7. The strain gauge-based load cells had a capacity of 50 kips (222 kN) each. These load cells were connected to a four-channel digital indicator that displayed the tendon force.

EXPERIMENTAL RESULTS

The tests were carried out by increasing the pressure in the hydraulic jack and recording the central deflection and the tendon forces at each increment of the load. The data for PT-a are provided in Table 1. The first crack in PT-a was observed at a load of 28.3 kips (125.9 kN) above the stub column face, as sketched in Fig. 7. The second perpendicular crack appeared at a load of 36.7 kips (163.2 kN). The specimen failed abruptly when it reached the maximum load of 38 kips (169 kN). The crack pattern at the top surface of PT-a at failure is shown in Fig. 8(a). The shape of the failure surface was roughly square with a dimension of approximately 18 to 22 in. (460 to 560 mm) on each side. Figure 8(b) shows the failure surface after the removal of the loose concrete. The results presented in Table 1 indicate that the tendon force increases linearly with the load from the decompression stage up to the point of failure. At the point of punching shear failure, the maximum load in the tendon was 19.7 kips (87.6 kN), which corresponded to a stress of 145 ksi (1000 MPa).

The central load versus deflection plots for PT-a and PT-b are provided in Fig. 9. The loaddeflection behavior appears nonlinear with the change in slope occurring prior to the first visible crack.

The data obtained for PT-c with a square column are provided in Table 2. The first crack in PT-c was observed at a load of 30.5 kips (135.7 kN) and the second perpendicular crack appeared at a load of 36.5 kips (162.4 kN). The maximum load reached was 43.2 kips (192.2 kN). The central load versus deflection plots for PT-c and PT-d are provided in Fig. 10. The load deflection behavior appears nonlinear as PT-a and PT-b.

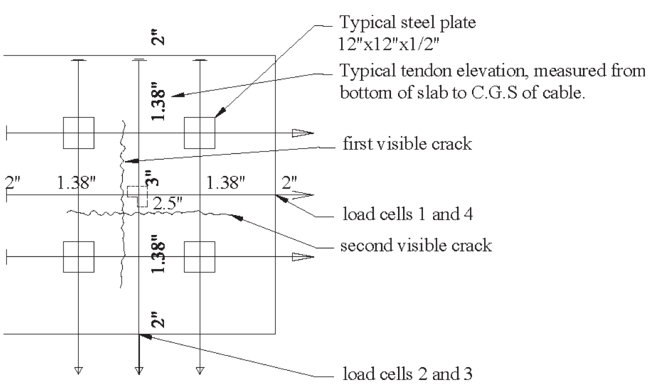

Fig. 7—Observed crack pattern in PT-a. (Note: 1 in. = 25.4 mm.)

DISCUSSION AND COMPARISON OF TEST RESULTS

The punching shear strength obtained from the current tests are compared with predictions obtained using equations proposed by previous investigators[3,4] and Eq. (5) in Table 3. A summary of the calculations follows (in.-lb units)

(1) Scordelis et al.[3] equation

$$V_c = (0.175 - 2.42 \times 10^{-5} f'_c + 2.0 \times 10^{-5} F_e/s) b_p d_{avg.} f'_c$$

Table 1—Test results for post-tensioned slab with L-shaped column (PT-a)

Load, lb	Deflection, in.	Tendon forces measured by load cells, lb				Remarks
0	0	17,711	16,622	20,820	18,703	—
2450	0.003	17,721	16,628	20,976	18,717	—
4640	0.009	17,723	16,634	20,980	18,722	—
7500	0.014	17,728	16,640	20,990	18,727	—
11,000	0.022	17,736	16,657	21,019	18,741	—
13,500	0.028	17,749	16,673	21,042	18,753	—
17,000	0.035	17,791	16,692	21,066	18,797	—
19,500	0.046	17,900	16,722	21,099	18,882	—
22,230	0.062	18,070	16,751	21,130	18,999	—
25,000	0.075	18,221	16,777	21,150	19,105	—
27,000	0.09	18,348	16,810	21,175	19,200	—
28,270	0.098	18,424	16,832	21,189	19,273	First visible crack
30,500	0.113	18,530	16,861	21,216	19,386	—
33,500	0.137	18,730	16,918	21,263	19,564	—
36,700	0.178	18,990	17,108	21,452	19,844	Second visible crack
38,000	0.196	19,720	17,723	22,108	20,437	Punching shear failiure

Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

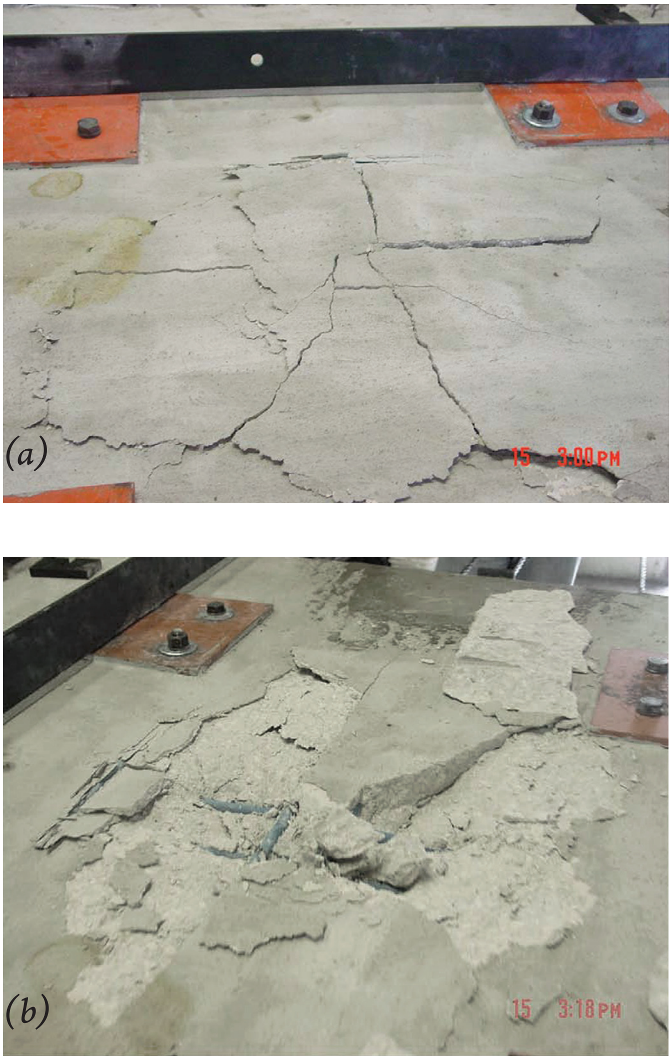

Fig. 8—Punching shear failure of PT-a: (a) cracking at tension face; and (b) top surface of slab after removal of loose concrete.

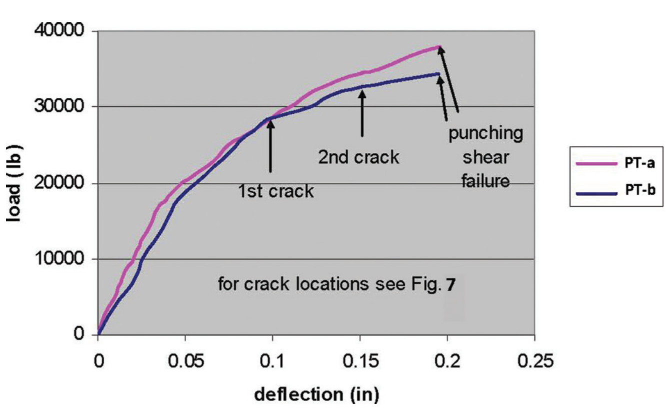

Fig. 9—Load-deflection diagram for PT-a and PT-b. (Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.)

Specimen PT-a and PT-b:

$$\frac{Fe}{s} = \frac{2 \times 18.4 \times 1000}{1.5 \times 12} = 2051 \frac{lb}{in.};$$

$$b_p = 14.82 \text{ in.}; d_{avg} = 2.75 \text{ in.}$$

$$V_c = (0.175 - 2.42 \times 10^{-5}(4250) + 2.0 \times 10^{-5}(2051))(14.82)(2.75)(4.25) = 19.6 \text{ kips}$$

Specimen PT-c and PT-d:

$$\frac{Fe}{s} = \frac{2 \times 18.8 \times 1000}{1.5 \times 12} = 2089 \frac{lb}{in.};$$

$$b_p = 16 \text{ in.}; d_{avg} = 2.75 \text{ in.}$$

$$V_c = (0.175 - 2.42 \times 10^{-5}(4350) + 2.0 \times 10^{-5}(2089))(16)(2.75)(4.35) = 21.4 \text{ kips}$$

(2) Grow-Vanderbilt[4] equation:

$$V_c = (0.360 + 0.3 f_{pc})bd$$

$$f_{pc} = \frac{6 \times 18.5}{72 \times 4} = 0.385 \text{ ksi}$$

Specimen PT-a and PT-b:

$$V_c = (0.360 + 0.3(0.385))(14.82)(2.75) = 19.4 \text{ kips}$$

Specimen PT-c and PT-d:

(3) Eq. (5):

$$V_c = (\beta_\varphi \sqrt{f'_c} + 0.3 f_{pc}) b_o d + V_p = v_c b_o d + V_p$$

Specimen	b_o, in.	d, in.	f_{pc}, psi	v_c, psi	V_p, kips	V_c, kips
PT-a or PT-b	27.07	3.2	385	343.7	5.47	35.2
PT-c or PT-d	28.80	3.2	385	346.4	5.47	37.2

Note: 1 in. = 25.4 mm; 1 psi = 0.006895 MPa; 1 kip = 4.448 kN.

The Scordelis et al.[3] equation and the Grow-Vanderbilt[4] equations were found to be very conservative compared with the test results. The average punching shear strength obtained for the nonrectangular columns (PT-a and PT-b) were found to be within 5% of the ACI 318 predictions,[8] and for the rectangular columns (PT-c and PT-d), the average punching shear strength was approximately 12% higher than that obtained using Eq. (5).[8]

Table 2—Test results for post-tensioned slab with rectangular column (PT-c)

Load, lb	Deflection, in.	Tendon forces measured by load cells, lb				Remarks
0	0	16,490	19,997	19,830	18,920	—
2600	0.002	16,491	19,998	19,831	18,922	—
5240	0.005	16,493	19,999	19,832	18,923	—
7860	0.0075	16,494	20,001	19,838	18,925	—
10,480	0.0105	16,496	20,005	19,844	18,935	—
13,440	0.0145	16,499	20,014	19,853	18,949	—
16,400	0.019	16,502	20,023	19,863	18,964	—
19,370	0.0245	16,510	20,042	19,879	18,988	—
22,330	0.032	16,527	20,070	19,899	19,013	—
25,070	0.0395	16,562	20,111	19,937	19,053	—
27,800	0.053	16,595	20,153	19,972	19,090	—
30,540	0.0775	16,665	20,216	20,010	19,131	First visible crack
33,280	0.1000	16,781	20,339	20,070	19,188	—
36,600	0.1375	16,983	20,556	20,137	19,263	Second visible crack
39,900	0.1795	16,289	20,899	20,356	19,504	—
43,210	0.2310	16,929	21,486	21,356	20,504	Punching shear failiure

Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

Table 3—Calculated shear strength

Specimen designation	Test results, kips (kN)	Average values, kips (kN)	L-S-M,* kips (kN)	G-V,† kips (kN)	Eq. (5), kips (kN)
PT-a	38.0 (169.0)	36.15 (160.8)	19.6 (87.2)	19.4 (86.3)	35.2 (156.6)
PT-b	34.3 (152.5)				
PT-c	43.2 (192.6)	41.50 (183.7)	21.4 (95.2)	20.9 (93.0)	37.2 (165.5)
PT-d	39.9 (177.9)				

*From Scordelis et. al[3] equation.
†From Grow-Vanderbilt[4] equation.

FURTHER RESEARCH

The current study was performed using a slab thickness of 4 in. (100 mm), which is smaller than the slab thicknesses used in practice. This has been the case with most of the previous investigations due to experimental limitations. Tests on slabs with higher thicknesses are needed to evaluate the size effect on the punching shear failure.

The specimens tested during the current investigation did not contain any bonded reinforcement in the cross-sectional area of the column strip to compare

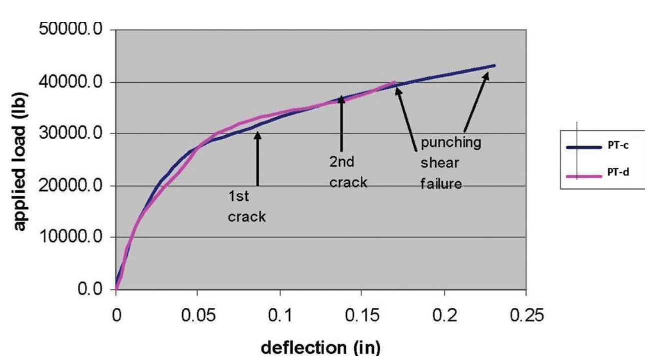

Fig. 10—Load-deflection diagram for PT-c and PT-d. (Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.)

the results with the current ACI 318 expression, which does not make any allowance for the presence of bonded reinforcement. Research is needed to incorporate the effect of bonded reinforcements in ACI 318 Code equation.

CONCLUSIONS

Results of punching shear of PT slabs with L-shaped columns were presented in this paper. The tests demonstrated that the PT slabs failed in punching shear. The average shear strength for square columns was 12% higher than that predicted by the ACI 318 equation, while it was only 3% higher than that predicted by the equations for the L-shaped columns. The lower margin of safety for the L-shaped columns should be investigated further and the expression for shear strength should be refined to obtain the same margin of safety for square and L-shaped columns.

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NOTATION

A_c = area of critical section = $b_o d$, in.^{2}
b_o = perimeter length of shear critical section, in.
b_p = perimeter of lifting collar, column, or column capital, in.
b_w = width of web of prestressed beam, in.
d = distance from the extreme compression fiber to the centroid of tension reinforcement, in.
F_e = effective prestress force per tendon, lb
= concrete compressive strength, psi
= concrete tensile strength, psi
f_{pc} = average compressive stress due to prestressing force after all losses, psi
s_t = tendon spacing, in.
V_c = punching shear load carried by concrete over entire critical section, kips
V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from principal tensile stress in web, lb
V_p = sum of vertical components of effective prestressing force for tendons crossing critical section, lb
v_{cw} = nominal shear stress, psi
x = distance between inflection points of tendon over column region, in.
y = distance from tendon inflection point to tendon high point, in.

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