Cyclic behaviour of interior post-tensioned flat plate connections

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In high seismic regions, post-tensioned (PT) slab–column frames are commonly used to support gravity loads in conjunction with a lateral-force resisting system (LFRS) such as a core wall. The LFRS is designed to resist 100% of the design lateral forces as well as to limit lateral displacements to an acceptable level, whereas the slab–column frame must sustain the gravity loads under the expected (design) displacements. Given the relatively sparse data on the seismic performance of PT flat plate slab–column frames, cyclic tests of four interior PT slab–column connections were conducted. Primary test variables were the level of gravity shear at the slab–column connection and the slab tendon arrangement. Test results indicate that both the test variables strongly influence the cyclic behaviour of the PT connections, and that the use of slab bottom reinforcement at the slab–column connection was effective in resisting positive moment developed under lateral loading as well as improving the hysteretic energy dissipation capacity.

Notation

\(A_{ps}\) area of tendons
\(A_{sm}\) area of continuous bottom bonded reinforcement
\(b_0\) perimeter of critical section
\(b_1\) width of critical section parallel to loading direction
\(b_2\) width of critical section perpendicular to \(b_1\)
\(c\) distance from the centroid of critical section to the perimeter of critical section
\(c_2\) column dimension parallel to the loading direction
\(d\) effective slab depth
\(d_b\) bar diameter
\(f_{pc}\) peak concrete compressive stress
\(f_{ps}\) nominal stress in an unbonded tendon (limited to lesser of \(f_{py}\) and \(f_{se} + 210\))

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forces and/or displacements is important. In high seismic regions (SDC-D or -E, IBC-03.1), where SDC indicates seismic design category; however, they may be utilised as intermediate moment frames (ACI 318–052 section 21.12.6) in areas with moderate seismic demands (SDC-A, -B and -C). Given the broad potential applications, a detailed understanding of slab-column frame behaviour subjected to lateral forces and/or displacements is important.

According to the seismic provisions in ACI 318–05, structural systems are either designated to resist earthquake forces [i.e. be part of the lateral-force resisting system (LFRS)] or they are referred to as ‘non-participating’ systems or ‘gravity’-force resisting systems (GFRS). In high seismic regions, post-tensioned slab–column frames are commonly used for GFRS, particularly for residential and office buildings where the LFRS consists of shear walls or moment-resisting frames at the building perimeter. When subjected to earthquake ground motions, the LFRS undergoes lateral deformations, which are imposed on the GFRS through the floor diaphragms. The lateral displacements imposed on the slab–column frame are likely to introduce significant unbalanced moments on the slab–column connections, increasing the potential for punching failures. Before the introduction of ACI 318–05,2 no specific requirements existed to avoid punching failures at slab–column connections in GFRS owing to the lateral displacement of the LFRS. However, the ability to maintain gravity loads after punching failure (i.e. post-punching resistance) could be justified owing to the ACI 3182 chapter 7 requirement for integrity reinforcement (continuous bottom reinforcement through the column cage).

According to the eccentric shear stress model in ACI 318–05,2 punching failures at slab–column connections subjected to shear (V₀) and unbalanced moment (M₀) occur where the shear stress owing to direct shear and eccentric shear exceeds the nominal shear stress on the slab critical section. The eccentric shear stress is a result of moment φM₀. The remaining portion of M₀ is assumed to be resisted by flexure by providing slab reinforcement over a slab flexural-thickness ratio of 35 to 45. Slab deflection control, and allow relatively large slab spans.

PT flat plate slab systems provide improved crack and flexible use of space. In particular, post-tensioned construction (e.g. slip forms), low floor-to-floor height and flexible use of space. In particular, post-tensioned (PT) flat plate slab systems are very efficient, since the PT flat plate slab systems provide improved crack and flexible use of space. In particular, post-tensioned flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient, since the PT flat plate slab systems are very efficient. Therefore, seismic performance of the PT connections, which tend to have relatively high gravity shear ratios owing to the substantial spans, have not been adequately studied. Furthermore, the impact of the tendon arrangement, which typically involves the use of banded tendons in one direction and distributed tendons in the other direction (ACI 423.3R-968), has not been systematically assessed, and the influence of bonded bottom reinforcement on the behaviour of PT flat plate systems subjected to moment reversal has not been addressed.

**Test programme**

**Prototype and specimen design**

Figure 1 depicts the prototype building selected to assist in the determination of specimen proportions and details. The prototype building is a ten-storey, post-tensioned flat plate slab system with 3.5-m storey heights with special reinforced concrete (RC) shear walls (R = 6) designed according to IBC-033 and ACI 318–052 requirements for SDC-E (high seismic risk). A slab thickness of 200 mm and a span length of 8 m were selected for the prototype building, resulting in a span-to-depth ratio of 40, which is within the range of 35 to 45 typically used for PT flat plate construction. The shear walls were designed to resist the lateral earthquake forces, whereas the PT flat plate slab systems were proportioned to support gravity forces.

Given that the prototype building is used for typical residential/office construction, the unit weight of the building is estimated as 23.5 kN/m³. Additional dead loads (e.g. partition walls) of 0.5 kPa and live loads of 2 kPa were chosen based on recommendations in IBC-03.1. Slab moments and shear forces owing to factored gravity loads were determined based on the results of an elastic analysis using MIDAS/Gen (MIDAS IT9 Version 6-3-2).

The specimens were approximately two-thirds scale representations of a typical interior slab–column connection within the prototype building (Fig. 1). Inflection points were assumed to occur at or near slab mid-span.
resulting in a span length of 4.6 m for the loading direction. A slightly shorter span length of 3.6 m was used in the transverse direction. A 300 × 300 mm column cross-section and a 130 mm slab thickness were selected based on the two-thirds scale factor.

Slabs were post-tensioned with 12.7 mm diameter, seven-wire strands with design yield stress of 1861 MPa. The arrangement of slab reinforcement is provided in Fig. 2. Minimum concrete clear cover was 12 mm for both top and bottom slab reinforcement. The post-tensioning strands were greased and placed in polyethylene tubes with a diameter of 16 mm; therefore, the tendons were ‘unbonded’. The number of post-tensioning tendons and the post-tensioning force per tendon were selected such that approximately 100% of slab dead weight was balanced (Table 1). The resulting average compressive stress \( f_{pc} \) of 1.21 MPa is within the allowable range of 0.88 MPa and 3.44 MPa as specified by ACI 318–05. According to section 18.9 of ACI 318–05 minimum bonded top reinforcement \( f_y = 352 \text{ MPa} \) was placed within an effective transfer width of \( c_2 + 3h \) at the connection region.

Although the slab–column frame is designed for gravity loads, it is subjected to the lateral deformations imposed on it by the lateral system (shear walls). To investigate the influence of the lateral deformations of the LFRS on the slab moment distribution, three-dimensional elastic analyses of combined PT flat plate frame and shear wall system were conducted for various levels of seismic demand. Resulting slab moments are depicted in Fig. 3. The slab moment diagrams indicate that the positive slab moment eventually develops on one side of the slab–column connection as the seismic demand is increased; therefore, sufficient bottom reinforcement should be provided to avoid the formation of large cracks. Requirements for minimum bonded bottom reinforcement for the post-tensioned connections do not exist; therefore, it is common to provide structural integrity reinforcement per ACI 318–05 section 7.13.2.5. This provision was developed primarily based on common practice for RC slab–column frames, which typically have slab span-to-thickness ratios closer to 20. Owing to the larger spans of typical of PT construction, bottom reinforcement satisfying ACI 3185 (section 7.13.2.5) and ACI-ASCE 35210 requirements [equation (1)] was provided.

\[
A_{sm} = \frac{0.5\phi \delta l_1 l_2}{\phi f_y}
\]

where \( \phi = 0.9 \). Accordingly, five D10 \( (d_b = 10 \text{ mm}) \) bottom bars were placed within a column width of \( c_2 \) for both directions, whereas no bottom reinforcement was provided outside of \( c_2 \).

**Material properties**

Normal Portland cement concrete with a design concrete strength of 29.8 MPa and a maximum aggregate size of 19 mm were used. The measured slump of the concrete ranged from 100 to 125 mm. As indicated in Table 2, mean compressive test-day strength of the concrete was 32.3 MPa based on compressive test results of five, 100 × 200 mm concrete cylinders in accordance with KS F 2404 (KS-0211). Seven-wire strands [steel wire for prestressed concrete (SWPC) 7B, \( f_y = 1555 \text{ MPa} \)] and D10 mild reinforcement were used for slab reinforcement, whereas D25 reinforcement \( (d_b = 25 \text{ mm}) \) was used for column longitudinal reinforcement. The SWPC 7B strands \( (f_s = 1555 \text{ MPa}) \) and reinforcement \( (f_y = 466 \text{ MPa}) \) were fabricated in accordance with KS D 7002 and KS B 0802, respec-
Table 1. Properties of specimens

<table>
<thead>
<tr>
<th>Mark</th>
<th>(c_2): mm</th>
<th>(d#): mm</th>
<th>(\rho_p): %</th>
<th>(f'_c): MPa</th>
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<td>32.3</td>
<td>1.21</td>
<td>343</td>
<td>82</td>
</tr>
</tbody>
</table>

\(c_2\): column dimension in the direction perpendicular to loading
\(d\#\): effective depth based on average of \(d_{ps}\) in two directions
\(\rho_p\): ratio of post-tensioning tendons
\(f'_c\): mean compressive concrete strength at the time of testing
\(f'_p\): average compressive stress in concrete owing to effective post-tensioning
\(V'_c\): nominal shear strength
\(V'_g\): gravity force to be transferred from slab to column
e.g. PI-B50, (P): PT, (I): interior, (B): banded, (50): \(V'_c/\phi V'_c \sim 0.50 \ (\phi = 0.75)\)

Testing and instrumentation

The four interior post-tensioned slab–column connections were subjected to uni-directional, reversed cyclic loading using the test set-up illustrated in Fig. 5. The primary variables of the test programme were the level of gravity shear, which could be varied using the hydraulic jack at the base of the column, and the arrangement of slab tendons (Table 1). For two of the specimens (PI-B50 and PI-B30), tendons were banded in the loading direction (E–W) and approximately uniformly distributed in the transverse direction (N–S), whereas for the other two specimens (PI-D50 and PI-D30), the tendons were uniformly distributed in the loading direction (E–W) and banded in the transverse direction (N–S), as shown in Fig. 2. The name PI-B50 refers to the following variables: Post-tensioned, Interior, Banded, with 50% gravity shear ratio. The gravity shear ratio is calculated as \(V'_g/\phi V'_c\), where \(V'_g\) is the factored gravity shear determined from analysis, \(V'_c\) is calculated according to ACI 318–052 (equation (11–36)), which includes the impact of the prestress, and \(\phi = 0.75\).

The base of the column was pinned, and the slab edges were pin-supported at the four corner points by struts (steel bars) with a diameter of 10 cm (Fig. 5). To evaluate the appropriateness of the boundary condi-
An elastic finite element analysis was conducted using MIDAS/Gen. This analysis showed that, for a given drift ratio, the difference in the unbalanced moments between the test condition (pinned at four corners) and the complete building system was less than 3%.

Gravity loads were simulated by applying an axial load at the base of the column using a 5000 kN hydraulic jack, as well as by placing loading blocks on the slab. Given that the ratio of shear force and moment at the critical section of the slab–column connection significantly influences the behaviour of the connections, the location of the loading blocks was determined based on an finite element analysis to match the ratio of shear force and moment for the specimens and the prototype building. As shown in Fig. 5, load cells were mounted on the hydraulic jack at the base of the column and on each of the four struts at the slab corners to monitor the applied loads.

The 250 kN capacity horizontal actuator was used to displace the top of the column (Fig. 5). A typical lateral displacement history consisting of three cycles at monotonically increasing drift levels between 0.2% and 6%, as shown in Fig. 6, was used for the tests. The applied lateral load and displacements were monitored using a load cell and linear variable differential transducers (LVDTs), respectively, and additional LVDTs were used to monitor (and avoid) slab twisting. Strain gauges were attached on selected bonded slab bars, and eight load cells were installed on tendons to measure the tendon forces during post-tensioning as well as changes in tendon forces during testing.

Table 2. Properties of materials

| Concrete | | | | | |
|---|---|---|---|---|
| $f'_c$: MPa | $\varepsilon_o$ | $E_s$: MPa | | |
| 32.3 | 0.00185 | 29 600 | | |

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<td>182 223</td>
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</table>

$f'_c$: mean compressive concrete strength at the time of testing

$f_y$: yield stress, $f_u$: ultimate stress

$\varepsilon_y$: mean strain at peak concrete strength, $\varepsilon_u$: ultimate strain

$d$: or $d$: rebar or strand diameter

$E_s$: steel modulus of elasticity

Fig. 4. Stress–strain relations of representative materials

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observed to extend approximately 300 mm (drift ratios greater than 3%, significant cracks were consequently extended across the entire slab width. For flexural cracks formed adjacent to the column and sub-sections, it is concluded that punching failures occurred away from the column face (Fig. 7). Based on observations, as well as the test data presented in the following subsections, it is concluded that punching failures occurred for all test specimens after flexural yielding of slab bonded reinforcement.

Lateral load versus lateral drift

Values of unbalanced moments obtained from two different approaches are plotted in Fig. 8. The first estimate of the unbalanced moment was obtained as the lateral load multiplied by a column height of 2.1 m, whereas the second estimate was derived from the difference of vertical reactions measured in the struts at the slab corners times the distance between the struts (4.8 m). A comparison of the results obtained using the two approaches is provided in Fig. 8 and indicates that consistent results were obtained.

Envelopes to the load–displacement relations as well as an idealised bi-linear relationship fitting to the envelope relations also are depicted in Fig. 9. The bi-linear relations are used to determine drift angles associated with yielding ($\theta_y$) and punching ($\theta_u$) as noted in Table 3. As depicted in Fig. 9, $\theta_y$ is defined as the drift ratio at punching ($\theta_{u,1}$) for specimens that experienced sudden punching failures (PI-B50, PI-B30 and PI-D50), or the drift angle at which the lateral load experiences a 20% drop from the peak lateral load ($\theta_{u,2}$) for the specimen PI-D30, which did not experience a sudden punching failure.

As noted for reinforced concrete connections, the limited data available have shown that lateral drift capacity at punching for post-tensioned slab–column connections is also strongly influenced by the magnitude of direct gravity shear stress applied on the critical section. A detailed review of the existing database for the PT connections indicates that higher drift ratios at punching were obtained for the PT connections, in part owing to the larger span-to-thickness ratios (40 to 45) compared with the RC connections ($\approx 25$), and that lower drift values were observed for the PT connections.
subjected to reversed cyclic loading relative to the PT connections subjected to monotonic or repeated lateral loading (Fig. 10). Since the database for PT interior connections subjected to reversed cyclic loading is limited to the four, isolated specimens with relatively high gravity shear ratios \( V_g/V_c = 0.46 \) to 0.72) tested by Qaisrani and Pimanmas et al., the test results presented for this study provide valuable information to assess trends for gravity shear ratios \( V_g/V_c \) between approximately 0.25 and 0.40 (actual values for the connections based on actual material properties, versus the design values of 0.30 and 0.50, respectively).

Figure 10 presents data \((V_g/V_c; \phi_u)\) for the four specimens tested (Table 3), along with the existing test data of post-tensioned slab–column connections without shear reinforcement. Based on the results presented in Fig. 10, a fairly consistent trend of decreasing drift ratio at punching \((\phi_u)\) for increasing gravity shear ratio \((V_g/V_c)\) is observed. The four test specimens from this study achieved substantially higher lateral drifts (3.2% to 5.9%) compared with earlier test results (1.8 to 2.3%) for interior connections subjected to cyclic loading with higher gravity shear ratios. Furthermore, specimens with lower gravity shear ratios (PI-B30 and PI-D30) achieved 180 and 135% of the lateral drift ratios of the companion

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bands tendons in one direction, may result in connec-
ductility for PI-B50 compared with PI-D50 is likely
proved ductility (190%) were observed for PI-D50
the moderate gravity shear ratio. However, for the high-
ratio, dissipated energy at failure differ substantially, as specimens
specimens with higher gravity shear ratios (PI-B50
and PI-D50), respectively, before punching failures
For PI-B30 and PI-D30, relatively large lateral drift
ratios were achieved for both specimens, 5.6 to 5.9%,
respectively, indicating that the tendon arrangement did
not impact the drift at punching failure significantly for
the moderate gravity shear ratio. However, for the higher
gravity shear ratio, based on the load-displacement
relations (Fig. 9), higher drift capacity (120%) and
improved ductility (190%) were observed for PI-D50
compared with PI-B50. The lower drift capacity and
ductility for PI-B50 compared with PI-D50 is likely
owing to larger precompression within the connection
region. In both cases of gravity shear ratios, larger drift
ratio at punching failure was achieved for the case with
distributed tendons. Current practice, which typically
bands tendons in one direction, may result in connec-
tions that have less drift capacity for loads parallel to
the banded direction.

**Dissipated energy**

The dissipated energy per cycle, which is used to
assess the hysteretic damping characteristics of the
system, was calculated based on the area of a load–
displacement relation for that cycle (Fig. 9). Based on
evaluation of these data, it is concluded that a gravity shear ratio \( V_g/\phi F_c \) has a modest impact of the accumu-
lated dissipated energy for specimens with the same
tendon distribution (for a given cycle). However, as
shown in Fig. 11, the values of the accumulated dissi-
pated energy at failure differ substantially, as specimens
PI-B30 and PI-D30 have values that are approximately
150% and 40% higher than those for PI-B50 and PI-
D50, respectively. The PT connections with higher
gravity shear (PI-B50 and PI-D50) tend to fail at a

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**Fig. 9. Lateral load plotted against lateral drift ratio**

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lower drift ratio, and thus achieve less ductility. In particular, yielding of the slab reinforcement of PI-B50 was limited prior to punching failure. These results indicate that the hysteretic energy dissipation capacity is dependent upon the gravity shear ratio.

Test data indicate that the hysteretic energy dissipation capacity also is sensitive to the tendon distribution pattern. Based on the relations plotted in Figs 9(a) and (b), specimens PI-B50 and PI-B30 exhibited fairly linear behaviour up to 2% drift with limited yielding of both top and bottom slab bonded reinforcement, probably as a result of high local precompression provided by the banded post-tensioning within the connection region. On the other hand, for specimens PI-D50 and PI-D30, a significant drop in lateral stiffness was observed at 1% drift (Figs 9(c) and (d)). As a result, the values of accumulated dissipated energy for PI-B50 and PI-B30 are approximately 90% and 40% of those for PI-D50 and PI-D30, respectively (Fig. 11).

Bonded slab reinforcement

As mentioned earlier, the test specimens included minimum bonded top reinforcement (eight D10 bars) in accordance with ACI 318–05 and bonded bottom reinforcement (five D10 bars) as required for structural integrity reinforcement based on ACI 352.1R-89.10 During the cyclic tests, both top and bottom bonded reinforcing bars reached the yield strain of approximately 0.002 (Fig. 12). Yield of top reinforcement initiated at drift ratios of 1.1, 1.7, 0.6 and 1.7% for PI-B50, PI-B30, PI-D50 and PI-D30, respectively, whereas yield of bottom reinforcement occurred between drift ratios of 2-2% and 3-5% for all specimens. Figures 12(a) and (b) show that, before punching, the degree of yielding of top reinforcement was more extensive than that of bottom reinforcement. Bottom reinforcement strains are plotted against the measured drift ratios in Fig. 13 for the bars located at the column centreline. Owing to gravity loading, bottom reinforcement was in compression before the application of lateral loading. The strains in bottom reinforcement began to vary from negative to positive values (i.e. moment reversal) at drift ratios of 0.8% and 0.5% for PI-B50 and PI-D50, respectively. This result is consistent with the observation that larger pre-compression within the connection region may have existed where banded tendons were parallel to the loading direction. For PI-B30 and PI-D30, with relatively low gravity shear stresses, moment reversal was observed at relatively low drift ratio of 0-5%.

In high seismic regions, the lateral (roof) drift demand on the structure is typically limited to 2% for life safety considerations,18 and some storey drift ratios typically exceed the roof drift ratio. Bonded bottom steel with the amount of $A_{sb}$ (equation (1)) is not anticipated to yield for drifts less than approximately 1-5%;14 however, moment reversal is likely to occur (Fig. 3). Therefore, bonded bottom reinforcement should be provided for PT flat plate systems designed

<table>
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<tr>
<th>Mark</th>
<th>Test results</th>
<th>Failure mode</th>
<th>Moment</th>
<th>Flexural strength</th>
<th>GSR: gravity shear ratio</th>
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<tr>
<td>PI-B50</td>
<td>$P_{peak}=43.6kN$</td>
<td>$\theta_{ub}=4.6%$</td>
<td>$\gamma_{M_{ub}}=11.1$</td>
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<td>$P_{peak}=47.8kN$</td>
<td>$\theta_{ub}=5.0%$</td>
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Fig. 10. Gravity shear ratio plotted against drift ratio at punching (PT slab–column connections)

Fig. 11. Accumulated dissipated energy

to resist gravity loads since tensile strains in the bottom slab reinforcement was observed before reaching a lateral drift ratio of 1.5% (Fig. 12(b)). The quantity of bottom reinforcement provided in the specimens was in compliance with equation (1) and was sufficient to resist positive moment developed up to lateral drifts of approximately 1.5% to 2.0% before reaching the yield strain (Fig. 12(b)), as well as to prevent progressive collapse. Accordingly, bottom reinforcement required by equation (1) is sufficient to limit extensive yielding; therefore, there does not appear to be a need to provide larger quantities of bottom reinforcement.

**Stresses in unbonded tendons**

For each specimen, average tendon stresses were monitored using seven or eight load cells mounted at

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tendon ends (TC, see Fig. 2). Figure 14 reveals that the stress in the unbonded tendons increased as a lateral drift ratio increased and that the tendon distribution influenced the change in the tendon stress in the loading direction. For the PI-B specimens, tendon stresses in the loading direction (TC 1, 2 and 3) tended to increase fairly uniformly irrespective of tendon location. However, for the PI-D specimens, only the stresses in tendons placed close to the column (TC 1 or 2) increased significantly (Fig. 14).

The peak values of the increase in the tendon stresses are in the range of 4-7 to 6-4% of the effective tendon stress ($f_{te}$). These values are substantially smaller than those ($\approx 15\%$) predicted by using equation (2) (ACI 318–05$^2$ equation 18–5)

$$f_{ps} = f_{te} + 70 + \frac{f_{te} \gamma_s}{300 \rho_p} \text{MPa} \quad (2)$$

The smaller increases may be partly attributed to the smaller total elongation of the tendon obtained for the interior connection under both gravity and lateral loads (plotted against the interior connection under gravity loads only, where the tendon in always in the tension zone). The lower stresses in the unbonded tendons under lateral loads result in reduced moment capacities under combined gravity and lateral load, relative to the nominal moment capacities computed for gravity load alone.

Shear strength of post-tensioned slab–column connections (ACI 318–05)

The shear strength of the specimens was evaluated using the eccentric shear stress model of ACI 318–05$^2$ chapter 11

$$\nu_u = \frac{V_u}{b_0 d} \pm \frac{\gamma_t M_{u,ub} c}{f_c} \leq \phi \nu_c \quad (3)$$

$$\gamma_t = \frac{1}{1 + (2/3)\sqrt{b_1/b_2}} \quad (4)$$

$$\gamma_v = 1 - \gamma_t \quad (5)$$

$$\phi \nu_c = \phi \left( \beta_p f_s + 3 f_{pc} + \frac{V_c}{b_0 d} \right) \quad (6)$$

where $\phi \nu_c$ is the nominal shear stress capacity of the post-tensioned connection without shear reinforcement reduced by the capacity reduction factor in units of MPa, $\beta_p$ is the smaller of 0.29 or $(\alpha_e/d_0 + 1.5)/12$ and $\alpha_e$ is 40 for interior columns.

In Table 3 column 10, the nominal moment for a slab width of $c_2 + 3h$ ($M_{u,c+3h} + M_{u,c+3h}$) is calculated for the specimens using the actual concrete strength ($f_s$), the actual yield stress of bonded bars ($f_y$), and the measured stress in unbonded tendons at failure ($f_{ps}$), given that all the slab bonded reinforcement yielded at the time of the failure. It is noted that the moment transferred by flexure ($\gamma_t M_{u,ub} \geq 0.6 M_{u,ub}$) and the flexural transfer capacity ($M_{u,c+3h} + M_{u,c+3h}$) are almost identical (Table 3, column 12), suggesting that the assumed fraction of unbalanced moment transferred by flexure ($\gamma_t = 0.6$) according to ACI 318–05$^2$ is reasonable for the PI-D specimens. For the PI-B specimens, most of post-tensioning tendons and all the bonded reinforcement were placed within $c_2 + 3h$ (Fig. 2). Thus, information provided in column 12 in Table 3 is not sufficient to assess the fraction of unbalanced moment transferred by flexure.

The peak values of the applied unbalanced moment ($M_{u,ub}$) monitored from a load cell attached to the horizontal actuator and the values of the total flexural moment capacity ($M_{u,c+3h} + M_{u,b}$) computed using measured material properties for the specimens are indicated.
in columns 4 and 11 in Table 3, respectively. Note that
\( M_{b,f,s} \) denotes the moment capacity considering full slab
width, not \( c_2 + 3h \). As might be expected, the values
obtained using the two different approaches are quite
close (ratios = 0.87 to 1.05), and also consistent with
other independently obtained measurements (load cells
installed at the base of slab end points) (see Fig. 9).
These results support that the shear stress capacities
(\( v_c \)) of the specimens were greater than the peak shear
stresses (\( v_{pc} \)) such that the full flexural moment capaci-
ties were reached prior to punching failures that even-
tually occurred at relatively large drifts (3.2 to 5.9%).

The peak shear stress (\( v_c \)) owing to direct shear and
eccentric shear obtained using equations (3) to (6) with
peak unbalanced moment (\( M_{u,ab} \)) was compared with
the nominal shear strength (\( v_c = 0.29 \sqrt{f_c'} + 0.3f_{pc} + V_p/\beta_d d \), where \( V_p \) is negligible) for the specimens in
column 8 in Table 3. Based on the results, the eccentric
shear stress model gives conservative results for the
nominal shear strength for the PT specimens, as well as
for the post-tensioned interior slab–column specimens
tested earlier (Fig. 15). Results for this test programme
are consistent with results for earlier test programmes
for gravity loads, although the ratios tend to be a bit
lower, possibly owing to the modest increase in tendon
stress under lateral loads noted earlier.

Conclusions

Experimental studies of four isolated, post-tensioned
interior slab–column connections subjected to both
gravity and cyclic lateral loading were conducted.
Based on the test results, the following conclusions are
reached.

(a) Consistent with observations for reinforced con-
crete slab–column connections, the level of gravity
shear on the slab critical section significantly influ-
ences the cyclic behaviour of the post-tensioned
slab–column connections. As the gravity shear ra-
tio increased, a drift ratio at punching and the
hysteretic energy dissipation capacity for the post-
tensioned connections decreased. The improved
hysteretic energy dissipation for the connections
with lower gravity shear (PI-B30 and PI-D30) was
owing to more extensive yielding of bonded rein-
forcement prior to punching failure.

(b) Results indicate that seismic performance of the
post-tensioned flat plate slab systems is impacted
by the tendon distribution. For the higher gravity
shear ratio (40%), higher drift capacity and im-
proved ductility were observed for PI-D50 com-
pared with PI-B50. For these cases, limited
yielding of both top and bottom slab reinforcement
was noted and the use of banded tendons appears
likely to create larger precompression within the connect-
ion region.

(c) Moment reversal (change from negative slab mo-
dent due to gravity load to positive slab moment
under lateral load on one side of a connection) occurred between lateral drifts of 0.5 to 0.8%. In
turn, bonded bottom reinforcement, which was
placed according to ACI 318–05 2 and ACI
352:1R-89, 10 reached yield at lateral drifts between
2.2% to 3.5%. Based on these results, it is con-
cluded that bonded bottom reinforcement should
be provided for the post-tensioned flat plate slab
systems; however, integrity reinforcement required
by chapter 7 of ACI 318–05 appears sufficient to
limit rebar yielding. In addition, the bottom rein-
forcement improves the hysteretic energy dissipa-
tion capacity of the PT interior connections.

(d) Test results indicate that tendon stresses for com-
bined gravity and lateral loading were approxi-
ately 35 to 60% of the values predicted by ACI
318–05 2 provisions. The reduced tendon stress
should be considered for flexural design of the
post-tensioned flat plate slab systems in high seis-
mic regions, as moment reversal is anticipated.

(e) The validity of the eccentric shear stress model
defined in ACI 318–05 2 was assessed. The test
results indicate that the eccentric shear stress mod-
el gives reasonable predictions for the nominal
shear strength (\( v_c \)) and the flexural transfer
capacity.

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