Effects of Bottom Reinforcement on Hysteretic Behavior of Posttensioned Flat Plate Connections

Sang Whan Han, Ph.D., P.E.1; Seong-Hoon Kee2; Young-Mi Park3; Sang-Su Ha, Ph.D.4; and John W. Wallace, Ph.D., P.E.5

Abstract: Flat plate systems are commonly designed as a gravity-force-resisting system (GFRS) along with stiffer lateral-force-resisting systems (LFRS) such as shear walls, and moment resisting frames; however, deformation compatibility between the GFRS and LFRS results in positive moments (producing tension at the slab bottom) at the slab-column connections in flat plate systems. Continuous slab-bottom reinforcement (integrity reinforcement) passing through the column is required to prevent progressive collapse; however, for posttensioned flat plate (PT-flat plate) systems, there have been arguments among engineers on applying continuous slab-bottom reinforcement since ACI 318 and 352 provisions do not clearly specify the requirement on the slab-bottom reinforcement for PT flat plates. The purpose of this study is to investigate the effects of continuous slab-bottom reinforcement passing through the column core on the hysteretic behavior of the PT slab-column connections. Test results for six, 3/5 scaled specimens for interior PT flat plate connections subjected to constant gravity loads and quasistatic reversed cyclic lateral loads are presented and indicate that the slab-bottom reinforcement conforming to ACI-ASCE 352R.1 is not required to prevent collapse, but does significantly improves the hysteretic energy absorption capacity.

DOI: 10.1061/(ASCE)0733-9445(2009)135:9(1019)

CE Database subject headings: Post tensioning; Slabs; Connections; Lateral forces; Plates; Reinforcement.

Introduction

Posttensioned flat plate (PT flat plate) systems are very effective due to the additional advantages of prestressed concrete, leading to reduction in construction time, greater span-to-depth ratios, and improved crack control compared with conventional reinforced concrete flat plate (RC flat plate) systems. In regions of high seismicity (such as Zone 3, 4 as defined in IBC (2006)], or for structures classified as high seismic performance or design category [such as seismic design category (SDC) D, E, F as defined in IBC (2006)], PT flat plate systems are commonly used as gravity force resisting system (GFRS). Stiffer lateral force resisting systems (LFRS) such as shear walls and moment resisting frames are used in conjunction with GFRS to provide lateral resistance to wind or earthquake loads. PT flat plates used as GFRS are commonly designed against gravity load, which produces negative slab moments (producing tension at slab top face) at slab-column connections. However, when subjected to lateral deformations under wind or earthquake loading, deformation compatibility between the LFRS and GFRS typically produces positive moments at slab-column connections (producing tension at slab-bottom face).

There are two significant differences between the reinforcement details used in RC and PT flat plate construction. For RC flat plate systems, minimum slab-bottom reinforcement is required for shrinkage and temperature according to ACI 318-08 Section 7.12, whereas for PT flat plates, the minimum slab-bottom reinforcement can be replaced with prestressed tendons if the tendons provide at least 0.7 MPa (100 psi) of average compressive stress (fpc) on the gross concrete section and the maximum spacing of tendons does not exceed the limits in ACI 318-08, Section 7.12.3. ACI 318 committee does not explicitly state the effect of slab-bottom reinforcement on shear capacity; however, continuous slab-bottom reinforcement is effective to increase the punching shear capacity of RC slab-column connections since this reinforcement increases shear friction along the slab critical sections, delays the higher extension of flexural slab cracks, and improves aggregation interlock and dowel action. For PT flat plates, similar shear friction resulting from precompression over the slab cross-section improves shear resistance of PT slab-column connection [ACI 318-08, Section 11.12.2.2; Han et al. 2006a,b].

In addition, for RC flat plate systems, continuous slab-bottom reinforcement passing though the column is required to prevent progressive collapse of slab-column connections (ACI 318-08, Section 13.3.8.5). For PT flat plates, there have been arguments among some engineers on applying continuous slab-bottom reinforcement since ACI 318 and 352 provisions do not clearly specify the requirement of the slab-bottom reinforcement for PT flat plates (ACI 318 Section 18.12.6). Moreover, this reinforce-
ment is sometimes omitted when providing continuous draped tendons in each direction through the critical section over columns (ACI 318 Section 18.12.6). Tendons passing through the slab-column joint at any location over the depth of the slab suspend the slab following a punching shear failure (ACI 318-08 R 18.12.6). These tendons have been shown to be effective in preventing progressive collapse (Mitchell and Cook 1984; Qaisrani 1993).

Experimental studies to investigate the hysteretic behavior of PT interior connections subjected to reversed cyclic lateral loads are very limited (Qaisrani 1993; Han et al. 2006a,b). In particular, the influence of slab-bottom reinforcement through the column core on the seismic performance of PT flat plate systems is unknown. Qaisrani (1993) reported that the direction of gravity induced moment could be reversed by the moment due to lateral loads (called as “moment reversal”) in the vicinity of the slab-column connections, indicating that positive moment induced by lateral loads surpassed the negative moment resulting from gravity loads. Recently, Han et al. (2006a,b) conducted experimental test of PT flat plate connection specimens to investigate the deformation capacity of PT interior flat plate connections. All specimens had continuous slab-bottom reinforcement through the column. It was observed that moment reversal due to lateral loads occurred at a drift ratios of 0.5–0.8%, and slab-bottom reinforcement through the column yielded at drift ratios of 2.5–3.2%.

Given the limited experimental data, it is difficult to evaluate the influence and importance of bottom reinforcement on the seismic performance of PT interior slab-column connections. Furthermore, among prior experimental studies, discrepancies exist in the test methods and the test variables, and the impact of slab-bottom reinforcement passing through the column core has not been systematically reviewed.

The influence of bottom reinforcement on the hysteretic behavior of PT flat plate connections is the focus of this study. Six approximately 3/5 scaled PT interior slab-column connection specimens were constructed and tested to evaluate the impact of slab-bottom reinforcement through the column, the level of gravity load, and tendon arrangement on the behavior of PT flat plate slab-column connections.

**Experimental Program**

**Design of Prototype and Specimens**

A typical ten-story office building was considered as the prototype for the design of the test specimens, which were designed according to ACI 318-08 [American Concrete Institute (ACI) Committee 318 (2008)] and IBC 2006. In this building, PT flat plate frames and shear walls were used as GFRS and LFRS, respectively. This building is assumed to locate in the region of high seismicity. Plan and elevation views of the prototype building are illustrated in Fig. 1. Design gravity loads on the slabs include dead load of 5.8 kPa calculated based on self-weight of structural members (=23.5 kN/m²), floor and partition weight (=0.49 kPa), and live load of 1.96 kPa specified in IBC 2006. Design seismic loads were calculated using the equivalent static lateral load procedure specified in IBC (2006). The SDC of the prototype building was assumed to be “E” producing a design base shear equal to 29% of the total building weight. Wind loads were estimated based on exposure category C. Basic wind speed was set to 40 m/s. Design member forces and moments were determined using the elastic finite-element analyses using the commercial software [MIDAS/GENw User’s Manual; version 6.3.2 (2004)].

Reinforcement details of the PT flat plate satisfied the requirements specified in ACI 318-08, Chapter 18 [American Concrete Institute Committee 318 (2008)].
Institute (ACI) Committee 318 (2008)]. The slab thickness of the prototype building was 22 cm, which resulted in a slab span-thickness ratio of 36.4. This thickness provides the sufficient shear strength against the shear force induced by gravity and moment due to the deformation compatibility between GFRS and LFRS (ACI 318-08, Section 21.13.6). Based on common practice, banded tendons were placed in one direction and uniformly distributed tendons were placed in the orthogonal direction (Fig. 2).

The unbonded tendons were prestressed to balance approximately 100% of total design dead loads so that average compressive stress in concrete ($f_{pc}$) was approximately 1.21 MPa (175 psi), which is within acceptable range of $f_{pc}$ of 0.86 to 3.44 MPa specified in ACI-ASCE 423 R.1 (ACI-ASCE Committee 423 1989).

Slab minimum top reinforcement equal to $0.0075 A_{cf}$ was placed within an effective slab width of $c_{2}+3h$ in accordance with ACI 318-08 (Section 18.9.3.3) [American Concrete Institute (ACI) Committee 318 (2008)], where $A_{cf}$ is the larger gross sectional area of the slab-beam strips in two orthogonal equivalent frames intersecting a column, and $c_{2}$ is the column width, and $h$ is the slab thickness. For three specimens (PI-B50X, PI-D50X, and PI-B70X), slab-bottom reinforcement through the column was not provided, whereas for three specimens (PI-B50, PI-D50, and PI-B70), the quantity of slab-bottom reinforcement through the column satisfied the ACI-ASCE 352 R.1 (ACI-ASCE Committee 352 1988) requirements for a minimum area of

$$A_{sb} = \frac{0.5w_{u}l_{2}}{\Phi f_{y}}$$

where $A_{sb}$=minimum area of effectively continuous bottom bars in each direction placed over the column; $w_{u}$=factored uniformly distributed load, but not less than twice the slab service dead load; $l_{1}$ and $l_{2}$=center-to-center span in each principal direction as defined in ACI 318–08; $f_{y}$=yield stress of steel; and $\Phi$=strength reduction factor of 0.9. Table 1 summarizes the dimensions and details of the specimens.

The PT flat plate specimens were approximately 3/5 scaled representations of the prototype slab-column connections between approximate slab midspans and column midheights as illustrated in Fig. 1(a). This test configuration is commonly used, as slab and column moments of prototype buildings are approximately zero at slab midspan and column midheight under lateral loading (i.e., plane frame assumption). Test slab plan dimensions for 3/5 scale were 4,800 mm × 4,800 mm in both directions; however, due to

### Table 1. Dimensions and Details of Specimens

<table>
<thead>
<tr>
<th>Mark</th>
<th>$c_{1}$=</th>
<th>$c_{2}$</th>
<th>$h$</th>
<th>$l_{1}$</th>
<th>$l_{2}$</th>
<th>$h_{1}$</th>
<th>$d_{ave}$</th>
<th>$\rho_{st}$ (%)</th>
<th>$\rho_{sb}$ (%)</th>
<th>$\rho_{st}$ (%)</th>
<th>$f_{pc}$ (MPa)</th>
<th>GSR (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI-B50</td>
<td>300</td>
<td>132</td>
<td>460</td>
<td>360</td>
<td>210</td>
<td>104</td>
<td>0.78</td>
<td>1.14</td>
<td>0.21</td>
<td>1.21</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>PI-D50</td>
<td>300</td>
<td>132</td>
<td>460</td>
<td>360</td>
<td>210</td>
<td>104</td>
<td>0.78</td>
<td>1.14</td>
<td>0.16</td>
<td>1.21</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>PI-B70</td>
<td>300</td>
<td>132</td>
<td>460</td>
<td>360</td>
<td>210</td>
<td>104</td>
<td>0.78</td>
<td>1.14</td>
<td>0.21</td>
<td>1.21</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>PI-B50-X</td>
<td>300</td>
<td>132</td>
<td>460</td>
<td>360</td>
<td>210</td>
<td>104</td>
<td>0.78</td>
<td>1.14</td>
<td>0.21</td>
<td>1.21</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>PI-D50-X</td>
<td>300</td>
<td>132</td>
<td>460</td>
<td>360</td>
<td>210</td>
<td>104</td>
<td>0.78</td>
<td>1.14</td>
<td>0.16</td>
<td>1.21</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>PI-B70-X</td>
<td>300</td>
<td>132</td>
<td>460</td>
<td>360</td>
<td>210</td>
<td>104</td>
<td>0.78</td>
<td>1.14</td>
<td>0.21</td>
<td>1.21</td>
<td>53</td>
<td></td>
</tr>
</tbody>
</table>

$^{a}$e.g., PI-B50X: (P)=PT; (I)=interior; (B)=banded; (50)=ratio of $V_{x}$ to design shear strength $\Phi V_{y}$ ($\phi=0.75$); and X=no bottom reinforcement; GSR (gravity shear ratio=$V_{x}/\Phi V_{y}$).
Material Properties

Normal-weight concrete, made from normal Portland cement, river sand, and gravel with a maximum size of 19 mm, were used for all specimens. The design strength of the concrete was 30 MPa. Five cylinder specimens, having a height of 200 mm and a diameter of 100 mm, were used to measure concrete compressive strength according to KS F2403 (Korean Standard Association 2002). The measured concrete strength at the time of testing ranged from 31.2 to 35.3 MPa, with mean compressive strength of 32.3 MPa. The peak stress was achieved at a strain of approximately 0.002 [Fig. 3(a)]. More detailed information on concrete mechanical properties is summarized in Table 2.

Bonded (deformed) reinforcement consisted of SD 40 with diameter of 9.5 mm (D10) and 25.4 mm (D25). Mean yield strengths based on three coupon tests for the D10 and D25 bars were 466 and 465 MPa, respectively. The tendons (SWPC 7B) used in the specimens were seven wire strands conforming to the requirements of KS D 7002 (Korean Standard Association 2002) with diameter of 12.7 mm. The tendons were covered with 16-mm diameter polyethylene tube coated with grease on the inner surface to prevent bond between tendon and concrete. The properties of reinforcing bars and tendons are summarized in Table 2, and the representative plots of measured stress and strain relation are given in Fig. 3(b).

Table 2. Material Properties

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Prestressing steel</th>
<th>Reinforcement rebar</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_c$ (MPa)</td>
<td>$E_{c, sec}$ (MPa)</td>
<td>$f_{pu}$ $^a$ (MPa)</td>
</tr>
<tr>
<td>32.3</td>
<td>0.00185</td>
<td>29,600</td>
</tr>
</tbody>
</table>

Note: $f'_c$=average compressive strength of concrete measured at the time of testing; $e_{cu}$=mean compressive concrete strain at peak strength; $E_{c, sec}$=secant elastic modulus of concrete; $E_i$, $E_{ps}$=elastic modulus of bonded reinforcement and of pretressing steel; $f_u$, $f_{pu}$=mean yield stress of bonded reinforcement and of prestressing steel; $e_{cu}$, $e_{pu}$=mean yield tensile strain of bonded reinforcement and prestressing steel; and $f_u$, $f_{pu}$=mean ultimate stress of bonded reinforcement and of prestressing steel.

$^a$f$_{pu}$ was based on 0.02% offset of initial stiffness.

Testing and Data Acquisition

The test frame with a specimen in position is shown in Fig. 4. The specimen boundary conditions were determined to simulate the boundary conditions for the prototype building under lateral loading, with four transducer struts located at the four corners of the slab. Hinges were installed at both ends of the strut to allow free rotation. A vertical oriented hydraulic jack between the base of the column and the strong floor (Fig. 4) was used to apply gravity load, while still allowing rotation. Twisting of the slab was prevented by out-of-plane support frames.

During the test, gravity loads were applied on the slab surface. The level of gravity load is expressed as the gravity shear ratio (GSR) ($V_g/V_c$), where $V_g$ is the gravity load applied at slab-column connection, and $V_c$ is the shear strength of specimens, and $\phi$ is the strength reduction factor taken equal to 0.75. The GSR for each specimen was selected such that $V_g$ was equal to 50 and 70% design shear strength ($\phi V_c$, $\phi=0.75$) to represent the cases of equivalent service dead load [1.0D] and factored dead load plus service live loads [1.2D+1.0L], respectively, for the prototype building. The location of the loading blocks was determined to properly simulate the actual moment-shear ratio that occurred at the connections in the prototype building estimated using elastic finite-element analyses [MIDAS/GENw User’s Manual; version 6.3.2 (2004)].

Cyclic lateral displacements were applied to the columns by a hydraulic actuator mounted horizontally between the column and the reaction wall to achieve the prescribed displacement loading history depicted in Fig. 5. All 61 channels of measured data were recorded by a computer data acquisition system. Applied loads were measured using four load cells attached at the end of transducer struts and one load cell fastened to the actuator. Stresses in...
the unbonded tendons were measured during test with loads cells located, as shown in Figs. 2(a and b). Concrete surface strains and reinforcement strains were measured by six and ten strain gauges, respectively.

**Experimental Results**

**Crack Patterns and Observed Damages**

A representative crack pattern at failure for all specimens is illustrated in Fig. 6. After gravity load was applied on the specimens, hairline flexural cracks were detected along the boundary between the column face and the slab surface. As lateral load increased, additional cracks formed and existing flexural cracks widened in the regions adjacent to slab-column connections. Subsequently, these cracks propagated to the sides of the column and diagonal (torsion) cracks (refer to (c) in Fig. 6) appeared, initiating at a top front corner of the column. The torsion cracks were detected at drift ratios of approximately 0.5, 0.5, and 0.35% for specimens PI-B50X, PI-D50X, and PI-B70X, and at drift ratios of 0.75, 0.75, and 0.5% for specimens PI-B50, PI-D50, and PI-B70. Drift ratio is defined as relative horizontal displacement between the top and bottom of the column divided by column (story) height. Radial torsional cracks (refer to (b) in Fig. 6) developed at a drift ratio of 0.5–0.75 and 0.75–1% for specimens without and with continuous slab-bottom reinforcement through the column, respectively. Subsequently, longitudinal and transverse flexural cracks widened, and extended to the edges of specimens with increasing lateral loads (refer to (a) and (d) in Fig. 6). Unlike the experimental results for RC slab-column connections (Pan and Moehle 1989; Robertson and Durrani 1992), for PT flat plates, torsional cracks were observed only in the vicinity of slab-column connection, as illustrated in Fig. 6. Specimens PI-B50, PI-D50, and PI-B70, with slab-bottom reinforcement, experienced punching shear failure at drift ratios of 3.3, 4.0, and 2.75%, respectively. Specimens PI-B50X, PI-D50X, and PI-B70X, without slab-bottom reinforcement, experienced punching shear failure immediately after flexural cracks formed on the slab-bottom face at drift ratios of 1.75, 2.75, and 0.75%, respectively.

Lateral loading cycles with increasing amplitude were applied until complete loss of lateral resistance of PT flat plate connections occurred. Residual strength ratio and plastic rotation of the specimens were estimated from Fig. 7 according to ASCE-41 (ASCE 2007) and are summarized in Table 3.

For Specimens PI-B50, PI-D50, and PI-B70, slab-bottom reinforcement and continuous draped tendons through the column continued to support gravity load after punching failure; therefore, collapse was not observed. Specimens without slab-bottom reinforcement through the column experienced punching shear failure earlier than specimens with slab-bottom reinforcement through the column (PI-B50X, PI-D50X, and PI-B70X); however, collapse was not observed for the maximum applied drift ratios due to the ability of the draped tendons passing through the col-
umn to support the gravity load after punching failure. This result suggests that the draped tendons are sufficient to avoid progressive collapse whereas slab-bottom reinforcement through the column reduced crack widths and increased the drift at punching failure substantially.

**General Behavior**

Measured lateral load versus lateral drift ratio for each specimen is plotted in Fig. 7. Yield and ultimate drift ratios ($\theta_y$ and $\theta_u$) are depicted in Fig. 7. Yield drift ratio ($\theta_y$) is defined as the drift ratio of slab-column connections, where the yield point is calculated based on an idealized bilinear relationship, as illustrated in Fig. 7 (Pan and Moehle 1989). Ultimate lateral drift ratio $\theta_u$ is determined as the drift ratio where the postpeak lateral load dropped to 80% of the peak lateral load. Fig. 7 also depicts the column lateral load ($V_{n, ACI}$); $V_{n, ACI}$ is obtained from $M_{n, ACI}$ divided by the distance between hinges at both ends of a column (=2.1 m); and $M_{n, ACI}$ is lesser of $M_{n, flexure}$ and $M_{n, shear}$, where $M_{n, flexure}$ is a moment calculated based on flexural capacity, and $M_{n, shear}$ is calculated based on punching shear strength. Note that, in this study, $M_{n, ACI}$ for all specimens were governed by $M_{n, shear}$, and more detailed description on calculation of $M_{n, shear}$ are available in the following section of “Shear stresses.”

**Discussion of Experimental Results**

**Deformability**

Important test results, such as peak lateral load, yield rotation, and ultimate rotation are summarized in Table 4. For similar specimens (i.e., either with or without bottom reinforcement), specimens with a higher GSR experienced punching failure at a lower drift ratio. For example, specimen PI-B50 and PI-B70, with banded tendons in the loading direction, achieved $\theta_u$ values of 3.3 and 2.75%, respectively. For specimens without bottom reinforcement, PI-B50X and PI-B70X, ultimate rotations were 1.75 and 0.75%. A similar trend was observed for the specimens having distributed tendons in the loading direction, PI-D50 and PI-D70, which had drift capacity of 4.0 and 2.75%, respectively.

The test results are consistent with results reported by others for RC connections (e.g., Pan and Moehle 1989; Hueste and Wight 1999; Roberson and Durrani 1992) and add substantial data to reinforce the results for PT connections subjected to cyclic loads reported by Kang (2004). In general, PT connections tend to achieve higher drift ratios than RC connections for the same GSR ratio.

Table 4 also summarizes the ratio of $\theta_u$ of specimens without slab-bottom reinforcement to $\theta_u$ of the corresponding specimens with slab-bottom slab reinforcement, with ratios of 0.27 (PI-B70), 0.53 (PI-B50), and 0.69 (PI-D50). These ratios indicate that specimens without slab-bottom reinforcement achieved substantially lower ultimate drift ratios at punching, and that the largest reduction occurred for the GSR ($=V_{g}/V_{c}$) of 0.7 where $\phi$ is 0.75. Comparing ultimate rotation values for specimens having the same GSR (0.5) with distributed and banded tendon arrangements (PI-B50 and PI-D50) indicates that use of continuous slab-bottom reinforcement has more impact on $\theta_u$ for the banded tendon arrangement (0.53 ratio versus 0.69). Specimen PI-B70X, with 70% GSR and no continuous bottom reinforcement through the column, experienced punching failure at a drift ratio slightly less than the drift limit specified in ACI 318-08, Section 21.11.5 (see Fig. 8) that requires the use of slab shear reinforcement. Note that the abscissa of Fig. 8 is expressed in terms of $V_{g}/V_{c}$ instead of $V_{g}/\phi V_{c}$ ($\phi$=0.75). GSRs 0.7 and 0.5 are equivalent to $V_{g}/V_{c}$ of 0.53 and 0.36, respectively. The other PT specimens had a drift capacity larger than the ACI limiting value. It is noted that the ACI limit value is approximately the mean minus one standard
deviation of the residuals for the available test data for RC connections (Kang 2004). Comparison with prior test results (Fig. 8) indicates that drift capacities of the specimens without bottom reinforcement are lower than the best fit line for PT connections subjected to reversed cyclic load history. For specimens PI-B50X and PI-B70X, the drift capacity approximately corresponds to the mean minus one-half standard deviation of the residuals from the best fit line of PT connections under cyclic loads, whereas for PI-D50X, the drift capacity was close to the mean minus one standard deviation of the residuals.

Deformability of the test specimens was affected by tendon arrangements. Specimens with distributed tendons placed in the loading direction provided improved drift capacity than corresponding specimens with banded tendon arrangement in the loading direction. For specimens with slab-bottom reinforcement, deformation capacities of PI-D50 increased 21% compared to that of PI-B50; similarly, for specimens without slab-bottom reinforcement, deformation capacity of PI-D50X improved 57% compared to that of PI-B50X.

Fig. 7. Lateral load versus lateral drift
Table 3. Comparison of Numerical Modeling Parameter

<table>
<thead>
<tr>
<th>Mark</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>a</th>
<th>b</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI-B70X</td>
<td>0.003</td>
<td>0.003</td>
<td>0.161</td>
<td>(0.0029)</td>
<td>0.0323</td>
<td>0.223</td>
</tr>
<tr>
<td>PI-B70</td>
<td>0.010</td>
<td>0.003</td>
<td>0.633</td>
<td>0.0085</td>
<td>0.0323</td>
<td>0.223</td>
</tr>
<tr>
<td>PI-B50X</td>
<td>0.006</td>
<td>0.006</td>
<td>0.252</td>
<td>0.0091</td>
<td>0.0091</td>
<td>0.000</td>
</tr>
<tr>
<td>PI-B50</td>
<td>0.013</td>
<td>0.161</td>
<td>0.349</td>
<td>0.016</td>
<td>0.0373</td>
<td>0.273</td>
</tr>
<tr>
<td>PI-D50X</td>
<td>0.010</td>
<td>0.010</td>
<td>0.349</td>
<td>0.0091</td>
<td>0.0091</td>
<td>0.000</td>
</tr>
<tr>
<td>PI-D50</td>
<td>0.026</td>
<td>N.A.</td>
<td>0.415</td>
<td>0.016</td>
<td>0.0373</td>
<td>0.273</td>
</tr>
</tbody>
</table>

Note: Values in parenthesis were based on “No” under the heading “Continuity Reinforcement” in ASCE/SEI-41.

Energy Absorption Capacity

Energy absorption capacity ($E_a$) is defined as cumulative energy until punching shear failure occurs, which is calculated using the following equation, and $E_a$ for each specimen is summarized in Table 4:

$$E_a = \sum_{i=1}^{N} E_i$$

where $E_i$ = energy absorption in each cycle $i$ of test and $N$ = cycle number when punching shear failure occurs.

As shown in Table 4, the energy dissipation capacity of specimens PI-D50, PI-B50, and PI-B70 were markedly higher than that of corresponding specimens PI-D50X, PI-B50X, and PI-B70X without continuous bottom reinforcement. The energy dissipation capacity of the specimen PI-D50X was roughly 50% of PI-D50 whereas specimens PI-B50X and PI-B70X achieved only 30 and 10% of the energy dissipation capacities of the companion specimens PI-B50 and PI-B70, respectively. Fig. 7 shows that the energy dissipation capacity tends to decrease with increasing gravity loads; the energy ratios for PI-B70 to PI-B50, and PI-B70X to PI-B50X are 58 and 33%, respectively. Fig. 7 shows that energy absorption capacity tends to decrease with increasing gravity loads; the energy ratios for PI-B70 to PI-B50 and PI-B70X to PI-B50X are 58 and 33%, respectively.

Table 4 shows that tendon arrangement in PT flat slabs affects energy dissipation capacity of PT flat plate specimens. Energy absorption capacity of the specimens (PI-D50 and PI-D50X) with distributed tendon arrangements in the loading direction is larger than that of the corresponding specimens (PI-B50 and PI-B50X) with banded tendon arrangements in the loading direction. The ratios for PI-D50 to PI-B50 and PI-D50X to PI-B50X are 162 and 255%, respectively.

Strain Distribution in the Vicinity of Columns

Data from strain gauges mounted on the slab concrete surfaces and on top and bottom slab reinforcement, along with tendon load cell (TC illustrated in Fig. 2) data, were used to detect moment reversal at the slab-column connection (Fig. 9). Moment reversal occurs when the moment direction induced by gravity loads is reversed by the moment due to lateral loads. For specimen PI-B50 [Fig. 9(a)], at a drift ratio of 0.5%, tensile strain was measured on the top reinforcement; and compressive strain on the bottom reinforcement, indicating that negative moment due to gravity loads exceeded the magnitude of the positive moment due to applied lateral load. Whereas for a drift ratio of 1.0%, compressive strain was detected on the top reinforcement and tensile strain was observed on slab-bottom reinforcement, indicating excess of the drift induced moment over that due to gravity loads. At a drift ratio of 3.5%, strain on the slab-bottom reinforcement exceeded the yield strain of 0.002. For specimen PI-B50X [Fig. 9(b)] compressive strains on top reinforcement and tensile strain on the slab-bottom surface were measured until a drift ratio of 1.0%. Moment reversal detected at a drift ratio of 1.4%. At a drift ratio of 1.75%, sudden punching shear failure occurred in specimen PI-B50X. Therefore, the test results clearly demonstrated that moment reversal occurred at the slab-column connection, creating tension on the bottom surface of the slab and the column.

Table 4. Summary of Test Results

<table>
<thead>
<tr>
<th>Mark</th>
<th>Ultimate lateral load (kN)</th>
<th>$\theta_y$ (%)</th>
<th>$\theta_m$ (%)</th>
<th>$E_a$ (kN m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>O</td>
<td>X</td>
<td>3=</td>
<td>2</td>
</tr>
<tr>
<td>PI-B70</td>
<td>34.4</td>
<td>26.7</td>
<td>0.36</td>
<td>1.4</td>
</tr>
<tr>
<td>PI-B50</td>
<td>43.6</td>
<td>21.6</td>
<td>0.50</td>
<td>2.1</td>
</tr>
<tr>
<td>PI-D50</td>
<td>49.1</td>
<td>26.7</td>
<td>0.54</td>
<td>1.4</td>
</tr>
</tbody>
</table>

#O and X indicate specimens with and without slab-bottom reinforcement through the column, respectively; $\theta_y$ = yield drift ratio; $\theta_m$ = drift ratio at punching shear failure; and $E_a$ = energy absorption capacity.
The portion of unbalanced moment transferred by flexure ($\gamma_f M_u$) and eccentric shear $(1 - \gamma_e) M_u$ were determined from the experimental results and compared with the values calculated according to ACI 318 Sections 13.5.3.2 and 11.12.6. The total unbalanced moment ($M_u$) was calculated by maximum lateral force ($V_{x, \text{max}}$) times the length between the two column hinges ($h_1$); $V_{x, \text{max}}$ is summarized in Fig. 7. The portion of the unbalanced moment transferred by flexure ($M_{f, c+3h}$) by the top and bottom reinforcement and unbonded tendons placed within the effective slab width ($c_3+3h$, ACI 318-08 Sec. 13.5.3.2) was calculated using Eq. (3a) for specimens with bottom reinforcement and Eq. (3b) for specimens without bottom reinforcement, based on coupling forces shown in Fig. 10. Note that force component of prestressing steel was considered to calculate the depth of equivalent rectangular stress block measured from extreme compressive fiber at front and back face (see Fig. 10), respectively; however, effects of prestressing steel in front face of the column was insignificant because of short moment arm, i.e., distance from centroid of prestressing steel to neutral axis [see Fig. 9(a)]; hence, it was ignored in the calculation of $M_{f, c+3h}$ in Eq. (3a). $M_{c+3h}$, cracking moment, was used to estimate the $M_{f, c+3h}$ for a specimen without slab-bottom reinforcement.

$$
M_{f, c+3h} = M_{f, c+3h} + M_{f, c+3h} = f_{p}A_{sp}d_p - \frac{a}{2} + f_{st}A_{st}d_{st} - \frac{a}{2}
$$ (3a)
M'_{f, c_2 + 3h} = M'_{f, c_2 + 3h} + M'_{f, c_2 + 3h}

\[ = f_{ps} A_p \left( d_p - \frac{a}{2} \right) + f_{st, b} A_{st} \left( d_{st} - \frac{a}{2} \right) + M'_{c} \]  

\[ \text{where } f_{ps} = \text{stress in prestressing steel at nominal flexural strength}; \]
\[ f_{st, b} \text{ and } f_{st, f} = \text{stresses of bonded reinforcement corresponding to strains } \varepsilon_{st, b} \text{ and } \varepsilon_{st, f} \text{ determined using the stress-strain curve in Fig. 3. Strain } \varepsilon_{st, b} \text{ is the strain measured on the slab top reinforcement parallel to the lateral loading in the back face of the column [see Fig. 10(a)], and } \varepsilon_{st, f} \text{ is the slab-bottom reinforcement in the front face of the column [see Fig. 10(a)], and } d_p, d_{st}, \text{ and } d_{sb} \text{ are distance from extreme compression fiber to centroid of prestressing steel, top reinforcement, and bottom reinforcement.} \]

Stress, \( f_{ps} \), was measured using donut-shaped load cells, denoted as TCs in Fig. 2. The variation of tendon stresses with drift ratio is shown in Fig. 11. In calculating \( M_{f, c_2 + 3h, \text{exp}} \), stress \( f_{ps} \) was estimated as the average value for the tendons located within \( c_2 + 3h \) in the loading direction. Fig. 12 shows strains on slab top reinforcement with respect to drift ratios. The fraction of moment transferred by flexure (\( \gamma_f \)) is given in Table 5 and was calculated as

\[ \gamma_f = \frac{M_{f, c_2 + 3h, \text{exp}}}{M_u} \]  

It is observed that \( \gamma_f \) does not vary according to the existence of slab-bottom reinforcement, which is consistent with the relatively small quantity of slab-bottom reinforcement. For PT flat plate slabs, moment transfer fraction (\( \gamma_f \)) is approximately 0.6, which is equivalent to \( \gamma_f \) for RC square interior flat plates with square columns without opening piercing the adjacent slab [American Concrete Institute (ACI) Committee 318-2008].

\[ v_u = \frac{V_u}{b_{st} d} + \frac{\gamma_f M_u c}{J} \]  

where \( V_u \) and \( M_u \) = shearing force and unbalanced moment at the centroid of the shear critical section, respectively, due to gravity loads combined with wind, earthquake, or other lateral forces; \( b_{st} \) = perimeter of shear critical section in slabs; \( J \) = polar moment of inertia; and \( c \) = centroid of shear perimeter. The fraction of

**Shear Stresses**

Shear stresses (\( v_u \)) at the slab-column critical section were determined based on the eccentric shear stress model specified in ACI 318-08 [Eq. (5)].
unbalanced moment transferred by shear (γ_u) for interior flat plate slabs having square columns was assigned 0.4 (≈1 − γ_f) in accordance with ACI 318-08, as well as per the results in the prior section.

Fig. 13 plots shear stresses versus drift ratio. Maximum shear stress for each specimen is summarized in Table 6. In Fig. 13, shear stress obtained using Eq. (5) is compared with nominal shear strength [Eq. (6)] obtained using ACI 318-08

\[
\nu_c = (\beta_s f_p/\gamma_f + 0.3 f_p) + \frac{V_v}{b_yd} \text{ (MPa)}
\]

where \( \beta_s \) = lesser value between 0.29 and 0.083(\( \alpha_d/d_y + 1.5 \)); \( \alpha_d \) = 40 for interior columns; and \( V_v \) = vertical component of all effective prestress forces crossing the critical section; but was ignored in the calculation of \( \nu_c \) in this study.

As shown in Fig. 13, specimens PI-B70, PI-B50, and PI-D50 with slab-bottom reinforcement through the column have maximum shear stresses exceeding the shear stress calculated by Eq. (5). However, specimens PI-B70X, PI-B50X, and PI-D50X without slab-bottom reinforcement had shear stress less than those calculated by Eq. (6). The results indicated that punching failures for specimens without bottom reinforcement occurred at shear stresses substantially below the ACI nominal shear strength, apparently due to the crack at the slab-column interface resulting from moment reversal.

Eq. (5) is rearranged to determine the maximum unbalanced moment producing punching failure as follows:

\[
M_u, \text{ ACI} = \left( \frac{\nu_u - V_u}{b_yd} \right) \frac{f}{\gamma_f c}
\]

The maximum unbalanced moment (\( M_u, \text{ exp} \)) obtained from experiment results for specimens without slab-bottom reinforcement does not reach the unbalanced moment (\( M_u, \text{ exp} \)) calculated using Eqs. (6) and (7), whereas \( M_u, \text{ exp} \) for specimens with slab-bottom reinforcement exceeds \( M_u, \text{ ACI} \). Table 6 summarizes unbalanced moments for the specimens obtained from the experiments and Eqs. (6) and (7).

### Table 5. Portion of Unbalanced Moment Transferred by Flexure (\( \gamma_f \))

<table>
<thead>
<tr>
<th>Mark</th>
<th>( \Delta f_{ps, \text{ exp}} ) (MPa)</th>
<th>( \Delta f_{ps, \text{ ACI}} ) (MPa)</th>
<th>( M_{f, \text{ unbal, exp}} ) (kN m)</th>
<th>( M_{u, \text{ exp}} ) (kN m)</th>
<th>( \gamma_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI-B50</td>
<td>59.6</td>
<td>108</td>
<td>56.96</td>
<td>91.6</td>
<td>0.62</td>
</tr>
<tr>
<td>PI-D50</td>
<td>58.1</td>
<td>145</td>
<td>55.0</td>
<td>99.3</td>
<td>0.55</td>
</tr>
<tr>
<td>PI-B70</td>
<td>6.5</td>
<td>108</td>
<td>43.5</td>
<td>72.3</td>
<td>0.60</td>
</tr>
<tr>
<td>PI-B50X</td>
<td>28.3</td>
<td>108</td>
<td>31.6</td>
<td>45.4</td>
<td>0.70</td>
</tr>
<tr>
<td>PI-D50X</td>
<td>4.7</td>
<td>145</td>
<td>30.2</td>
<td>56.1</td>
<td>0.54</td>
</tr>
<tr>
<td>PI-B70X</td>
<td>12.9</td>
<td>108</td>
<td>16.7</td>
<td>25.8</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Note: \( \Delta f_{ps, \text{ exp}} \) = measured stresses increments in prestressing steel at the maximum strength of the tests; and \( \gamma_f \) = portion of unbalanced moment transferred by flexure.

After yield; \( c \) is to measure the reduced resistance after sudden reduction from \( C \) to \( D \), ASCE/SEI-41 (Tables 6–14) specifies magnitude of \( a \), \( b \), and \( c \) with respect to the ratio of gravity shear (\( V_g \)) acting on the slab critical section to direct punching shear strength (\( V_c \)), and existence of continuity reinforcement.

For comparison purpose, plastic rotation angles (\( a \) and \( b \) in Fig. 14) and residual strength ratio (\( c \) in Fig. 14) were estimated using the ASCE-41 requirements using linear interpolation based on GSR for test specimens (0.7 and 0.5). As mentioned earlier, GSRs of 0.7 and 0.5 are equivalent to \( V_g/V_c \) of 0.53 and 0.38, respectively. Results are summarized in Table 3 for both, experimentally observed parameters, \( a \), \( b \), and \( c \) as well as interpolated values for \( a \), \( b \), and \( c \) using the ASCE 41 (Tables 6–14).

As shown in Table 3, plastic rotations and residual strength ratio for the specimens with slab-bottom reinforcement passing through the column (i.e., PI-B50, PI-D50, and PI-B70) were greater than or slightly less than those specified in ASCE-41 (ASCE 2007) for PT connections having continuity reinforcement, whereas for the specimens without slab-bottom reinforcement passing through the column (i.e., PI-B50X, PI-D50X, and PI-B70X), plastic rotations and residual strength ratio were much less than those specified in ASCE-41 for PT connections with continuity reinforcement; however they are close to those for PT connections without continuity reinforcement. Fig. 14 compares the modeling parameters specified in ASCE-41-S1 with those for specimens PI-D50 and PI-D50X. It is worthy noting that ASCE-41-S1 provides the same modeling parameters \( a \), \( b \), and \( c \) for PT slab-column connections, if there is at least one of the posttensioning tendons in each direction passing through the column cage regardless of existence of continuous bonded reinforcement.

### Conclusions

This study focused on investigating the effect of slab-bottom reinforcement through the column on the hysteretic behavior of PT flat plate systems. Based on large-scale tests of six specimens, the following conclusions were reached:

1. PT slab-column connections with slab-bottom reinforcement through the column achieved larger deformation, and energy dissipation capacity, and greater strength than corresponding specimens without slab-bottom reinforcement through the column. Deformation capacity and energy absorption capacity ratios of PT connections with slab-bottom reinforcement to PT connections without slab-bottom reinforcement are as large as 3.67 and 10.5 (PI-B70 versus PI-B70X), indicating that bottom reinforcement has a significant impact on connection behavior. PT specimens without bottom reinforcement through the column achieve drift capacity at punching
that is similar to test results for RC slab-column connections. It is noted that specimen PI-B70X with relatively high gravity shear had a drift capacity less than a limiting value specified in ACI 318-08 (Fig. 8). Given these results, it is recommended that continuous slab-bottom reinforcement through the column shall be placed in PT slab-column connections.

2. The level of gravity loads was a pivotal factor on the hysteretic behavior for PT flat plate connections, for both specimens with and without slab-bottom reinforcement. With increasing GRS, strength, energy absorption capacity, and deformation capacity decreased substantially. For example, maximum lateral strength, energy absorption capacity, and deformation capacity of specimen PI-B70 (GRS=70%) are 21, 16, and 41% less than those of specimen PI-B50 (GRS=50%). The similar observation was made for the specimens without slab-bottom reinforcement passing through the column (PI-B50X and PI-B70X).

3. The hysteretic behavior of the test specimens was impacted by the tendon arrangement. Specimens with distributed tendons placed in the loading direction provided better hysteretic behavior than corresponding specimens with banded...
tendon arrangement in the loading direction. For specimens with slab-bottom reinforcement, energy dissipation capacities and deformation capacities of PI-D50 increased 21 and 62% compared to that of PI-B50; similarly, for specimens without slab-bottom reinforcement, dissipation capacity and deformation capacity of PI-D50X improved 57 and 156% compared to those of PI-B50X.

4. For PT slab-column connections, the fraction of unbalanced moment transferred by flexure is not substantially impacted by the presence of continuous slab-bottom reinforcement.

5. Specimens without slab-bottom reinforcement achieved approximately 75% of the nominal shear strength when punching shear failure occurred; though, progressive collapse was not observed for any specimens due to the existence of the continuous draped tendons through the column.

6. Residual strength ratio and loss of gravity load capacity parameters defined in ASCE-41 for the specimens without slab-bottom reinforcement passing through the column cage (PI-B50X, PI-D50X, and PI-B70X) were lower than those for the specimens with continuous reinforcement (PI-B50, PI-D50, and PI-B70). For specimens PI-B50X, PI-D50X, and PI-B70X, estimates for c and b parameters were significantly less than those specified for PT slab-column connections with continuity reinforcement by ASCE/SEI 41, whereas these estimates were close to those for PT slab-column connections without continuity reinforcement. Thus, numerical modeling parameters for PT slab-column connections in ASCE/SEI 41 have to be specified according to the existence of continuous slab-bottom reinforcement.

### Notation

The following symbols are used in this paper:

- $A_{cf}$: larger cross-sectional area of slab-beam strips in flat plate systems;
- $A_i$: area surrounded by the $i$th cycle of the lateral load and lateral drift relation;
- $A_{ab}$: minimum area of effectively continuous bottom bars in each direction placed over the column;
- $b_o$: perimeter of shear critical section in slabs;
- $c$: distance between centroid of shear critical section and the perimeter of critical section;
- $c_2$: column dimension perpendicular to loading direction;
- $d_{ave}$: effective slab depth of tendons based on average $d_p$ in two directions;
- $d_p$: distance from extreme compression fiber to centroid of prestressing steel;
- $f_{se}$: distance from extreme compression fiber to centroid of bottom reinforcement;
- $d_{st}$: distance from extreme compression fiber to centroid of top reinforcement;

### Table 6. Comparison between Experimental and Analytical Results

<table>
<thead>
<tr>
<th>Mark</th>
<th>$v_u$ (MPa)</th>
<th>$v_c$ (MPa)</th>
<th>$v_u/v_c$</th>
<th>$M_u$, exp (kN m)</th>
<th>$M_u$, flexure (kN m)</th>
<th>$M_u$, shear (kN m)</th>
<th>$M_u$, ACI (kN m)</th>
<th>$M_u$, exp/$M_u$, ACI</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI-B70-X</td>
<td>1.46</td>
<td>2.01</td>
<td>0.72</td>
<td>25.8</td>
<td>89.2</td>
<td>60.9</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>PI-B50-X</td>
<td>1.51</td>
<td>2.01</td>
<td>0.75</td>
<td>45.4</td>
<td>89.2</td>
<td>80.2</td>
<td>0.57</td>
<td></td>
</tr>
<tr>
<td>PI-D50-X</td>
<td>1.61</td>
<td>2.01</td>
<td>0.80</td>
<td>56.1</td>
<td>89.6</td>
<td>80.2</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>PI-B70</td>
<td>2.23</td>
<td>2.01</td>
<td>1.09</td>
<td>72.3</td>
<td>105.7</td>
<td>60.9</td>
<td>1.19</td>
<td></td>
</tr>
<tr>
<td>PI-D50</td>
<td>2.52</td>
<td>2.01</td>
<td>1.25</td>
<td>99.3</td>
<td>104.1</td>
<td>80.2</td>
<td>1.24</td>
<td></td>
</tr>
</tbody>
</table>

Note: $M_u$, exp = $\frac{V_{max} h_1}{h_2}$; $M_u$, ACI is the lesser of $M_u$, flexure and $M_u$, shear; and $v_c$ = shear stress calculated using Eq. (6) without considering the $V_p$ component.

### Fig. 14. Comparison of numerical modeling parameters
\[ E_u = \text{accumulated energy absorption capacity of a slab-column connection}; \]
\[ E_{esec} = \text{secant elastic modulus of concrete}; \]
\[ E_{ps} = \text{elastic modulus of prestressing steel}; \]
\[ f'_c = \text{compressive strength of concrete measured at the time of testing}; \]
\[ f_{pc} = \text{compressive stress in concrete (after allowance for all prestress losses) at centroid of cross-sectional resisting externally applied loads}; \]
\[ f_{ps} = \text{stress in prestressing steel at nominal flexure strength (limited to lesser of } f_{ps} \text{ and } f_{se} + 210); \]
\[ f_{py} = \text{yield stress of prestressing steel}; \]
\[ f_{se} = \text{effective stress in prestressing steel}; \]
\[ f_y = \text{yield stress of bonded reinforcement}; \]
\[ h = \text{the slab thickness}; \]
\[ J = \text{polar moment of inertia of shear critical section}; \]
\[ l_1 = \text{length of span parallel to the loading direction, measured center to center of supports}; \]
\[ l_2 = \text{length of span perpendicular to } l_1, \text{measured center to center of supports}; \]
\[ M_f = \text{unbalanced moment transferred by flexure}; \]
\[ M_{f,c2+3h} = \text{moment for an effective slab width of } c_2+3h \text{ based on measured material properties}; \]
\[ M_{n,ACI} = \text{nominal unbalanced moment lesser of } M_{n,\text{flexure}} \text{ and } M_{n,\text{shear}}; \]
\[ M_{n,\text{flexure}} = \text{nominal moment causing flexural-dominated failure}; \]
\[ M_{n,\text{shear}} = \text{nominal moment causing shear-dominated failure}; \]
\[ M_u = \text{unbalanced moment at the centroid of the shear critical section due to gravity loads combined with wind, earthquake, or other lateral forces}; \]
\[ M_{u,\text{exp}} = \text{unbalanced moment at the centroid of the shear critical section at maximum strength (} = V_{max} \times h_1); \]
\[ M_s = \text{unbalanced moment transferred by eccentricity of shear}; \]
\[ V_c = \text{nominal concrete shear strength}; \]
\[ V_g = \text{gravity force transferred from a slab to column}; \]
\[ V_{n,ACI} = \text{base shear of column associated with the nominal unbalanced moment}; \]
\[ V_p = \text{the vertical component of all effective prestress forces crossing the critical section}; \]
\[ V_s = \text{shearing force at the centroid of the shear critical section, due to gravity loads combined with wind, earthquake, or other lateral forces}; \]
\[ w_p = \text{factored uniformly distributed load, but not less than twice the slab service dead load}; \]
\[ \gamma_f = \text{factor used to determined the unbalanced moment transferred by flexure}; \]
\[ \gamma_v = \text{factor used to determined the unbalanced moment transferred by eccentricity of shear}; \]
\[ \Delta f_{ps,\text{exp}} = \text{measured stress increments in prestressing steel at the maximum strength of the tests}; \]
\[ \varepsilon_c = \text{mean compressive concrete strain at peak strength}; \]
\[ \varepsilon_{py} = \text{yield tensile strain of nonprestressing reinforcement}; \]
\[ \varepsilon_y = \text{yield tensile strain of prestressing reinforcement}; \]
\[ \theta_f = \text{drift ratio of slab-column connection at punching (} = \text{ultimate drift ratio}); \]
\[ \rho_s = \text{ratio of cross-sectional area of tendon in the slab effective width to } h(c_2+3h); \]
\[ \rho_{as} = \text{ratio of cross-sectional area of bottom rebar in the slab effective width to } h(c_2+3h); \]
\[ \rho_{ps} = \text{ratio of cross-sectional area of top rebar in the slab effective width to } h(c_2+3h); \]
\[ \Phi = \text{strength reduction factor [0.9 for Eq. (1), and 1 for GRS]}. \]

Acknowledgments

The writers acknowledge the financial supports provided by the Korea Research Foundation (Grant No. D01140) and SRC/ERC (Grant No. R11-2005-056-04002–0). The views expressed are those of authors, and do not necessarily represent those of the sponsors. The comments by the anonymous reviewers are greatly acknowledged.

References


