SEISMIC PERFORMANCE OF POST–TENSIONED INTERIOR SLAB-COLUMN CONNECTIONS WITH AND WITHOUT DROP PANEL

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ABSTRACT :

This paper presents an experimental study on the seismic performance of two three-fifth scale post-tensioned (PT) interior slab-column connection models, one without drop panel and another one with drop panel. The model without drop panel was designed and constructed to represent a typical connection between interior column and post-tensioned flat plate with bonded tendons usually found in Thailand. The other model was intended to represent an improved design of typical post-tensioned slab-column connections by using drop panel. Both models were tested under a constant gravity load. A conventional displacement-controlled cyclic loading routine with monotonically increasing drift levels until failure was adopted to investigate the seismic performance. The experimental results indicated that each model behaved like a linear elastic system with low energy dissipation, and no pinching was observed in the hysteretic loops. The model without drop panel abruptly failed by punching shear shortly after attaining its maximum lateral strength at 2.0% drift, while the improved model experienced a saturation of lateral peak load from about 2.5% to 6.0% drift prior to punching shear. The improved model with drop panel showed considerably more drift capacity and ductility than the one without drop panel. The test results also suggest that the gravity shear ratio be a major variable which governs the drift capacity and ductility of bonded PT interior connections, and that the ACI 318-05's design drift limit could be used for this case.

KEYWORDS: Slab-column connections, drop panel, post-tensioning, flat plate, punching shear, bonded tendons.

1. INTRODUCTION

Post–tensioned (PT) flat plate construction has long been popular in Thailand for medium–rise to high–rise buildings such as office buildings, hospitals, residential buildings, university, and parking buildings. As a general design practice, flat plate structures are designed primarily for gravity loads, while they are coupled with concrete shear walls for resisting lateral wind load. The flat plate structures are normally neither designed for lateral seismic load nor checked for deformation compatibility with the shear walls to ensure their ability to undergo the maximum earthquake–induced lateral drift without losing of the gravity load carrying capacity.

It is widely known that the slab–column connection is a critical component in the flat plate system. Under a strong earthquake ground motion, brittle punching shear may occur in this critical zone due to a combination of direct gravity shear and eccentric shear from earthquake–induced unbalanced moment between slab and column. In addition, extensive cracks in the connection region caused by repeated reversals of large lateral drift may result in a significant deterioration of the shear capacity of the connection. The punching shear failure may lead to a progressive collapse of the whole flat plate building.

Although numerous experimental studies on the seismic performance of slab–column connections have been carried out over the past two decades, most of these works focused on the seismic response of reinforced concrete (RC) flat plates. A limited number of studies investigated the seismic capacity of PT flat plates (Hawkins 1981, Foutch et al. 1990, Qaisrani 1993, Martinez–Cruzado et al. 1994, Kang and Wallace 2006, Gayed and Ghali 2006). All tested PT specimens were hitherto made to represent unbonded flat plate connections.

This paper presents the results of a series of tests performed on three–fifth scaled bonded PT slab–column connections with non–seismic detailing at the Structural Engineering Laboratory at Asian Institute of Technology, Thailand. Two interior connections, one without drop panel and the other with drop panel, were subjected to a quasi–static cyclic loading routine. The connection without drop panel, tested earlier by Warnitchai et al. (2004), was carefully designed and constructed to represent typical PT slab–column connections in Thailand. The test results of the connection without drop panel showed that the connection abruptly failed by punching shear after reaching 2% drift. Since drift at punching was considered rather low, a design improvement for the connections was proposed in this study. The connection with drop panel, which was tested in this study, was intended to represent an improved design of the connections without drop panel. This paper will discuss about the seismic performance and the effectiveness of incorporating drop panel in enhancing the seismic capacity of bonded PT interior flat plate connections.

2. EXPERIMENTAL PROGRAM

2.1. Description of Specimens

Two specimens were constructed to represent the bonded post–tensioned (PT) slab–column interior connections, one of which was without drop panel and the other was with drop panel. The specimen without drop panel was modeled after typical connections found in most PT flat plate buildings in Thailand. The PT flat plate buildings in Thailand are usually designed with 200 mm slab thickness, 8000 mm spans, 3000 mm story heights, and 400x800 mm rectangular columns. Generally, the flat plate floors are expected to act as the gravity load resisting system only. The design gravity loads in slabs consist of slab self weight and 250 kg/m^2 live load. On the other hand, the specimen with drop panel was modeled after the specimen without drop panel. The specimen with drop panel commonly followed the typical detail and loading of the specimen without drop panel. The drop panel is a third of slab span in length and 1.67 times slab thickness in depth. The conventional reinforcing bars in the specimen with drop panel were provided and placed in such a way that the respective specimen may have better seismic performance than that of the specimen without drop panel. Full details on the reinforcement layout of both specimens will be later explained in this section.

Both specimens were constructed at three–fifth scaled of the typical flat plate structures in Thailand. Thus, the slabs were all 5000 mm spans, one of which was without drop panel and the other was with 1600 mm square drop panel. The thickness of slab and drop panel was 120 mm and 200 mm, respectively. The size and height of column in both specimens were 250x500 mm and 1800 mm, respectively. The specimen with drop panel, which was tested in this study, is shown in Fig. 1. As the inflection points in the interior connection under seismic loading are assumed to occur at column mid–height and slab mid–span, the column of both specimens extended above and below the slab to story mid–height. Meanwhile, the slab of both specimens extended to mid–span on the two parallel sides of the slab–column connection. The column bottom end of both specimens was pinned and the column top end was connected to a hydraulic actuator through a pivoted connection. The hydraulic actuator simulated a lateral point load applied to the column top end of both specimens. Both slabs were supported along each transverse edge by 5 pin–ended bars to simulate a moment–free boundary condition. On all sides of the slabs, the edge beam with sufficient reinforcing bars was constructed to carry the high compressive load induced by prestressing tendons at the anchorages.

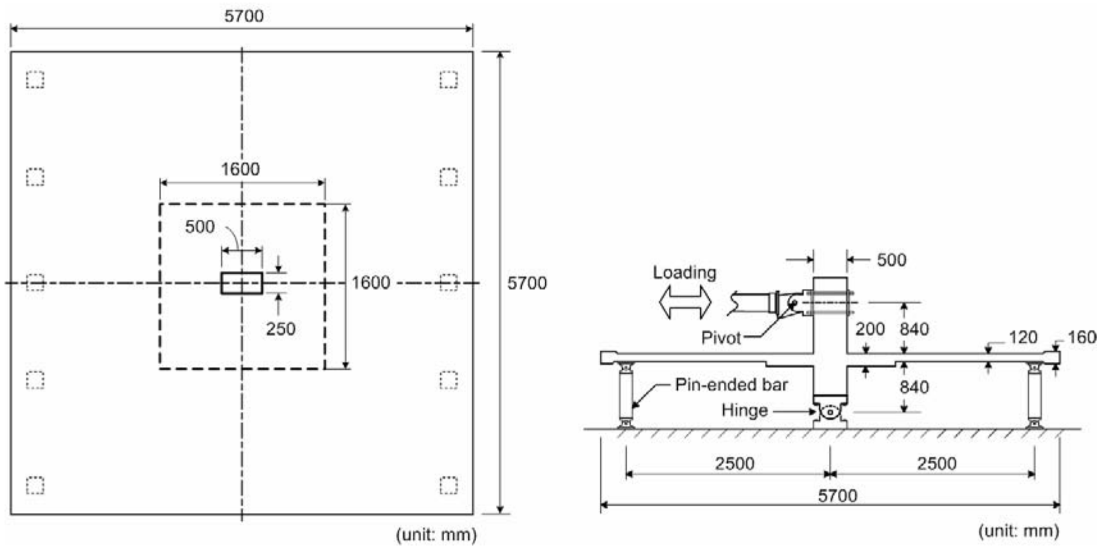
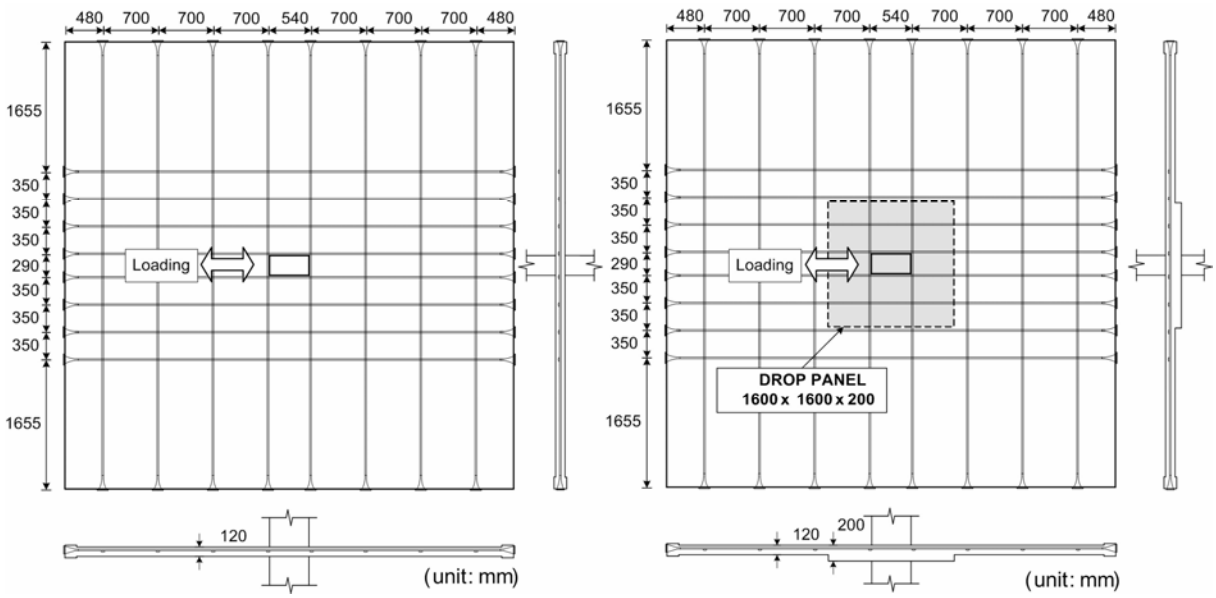


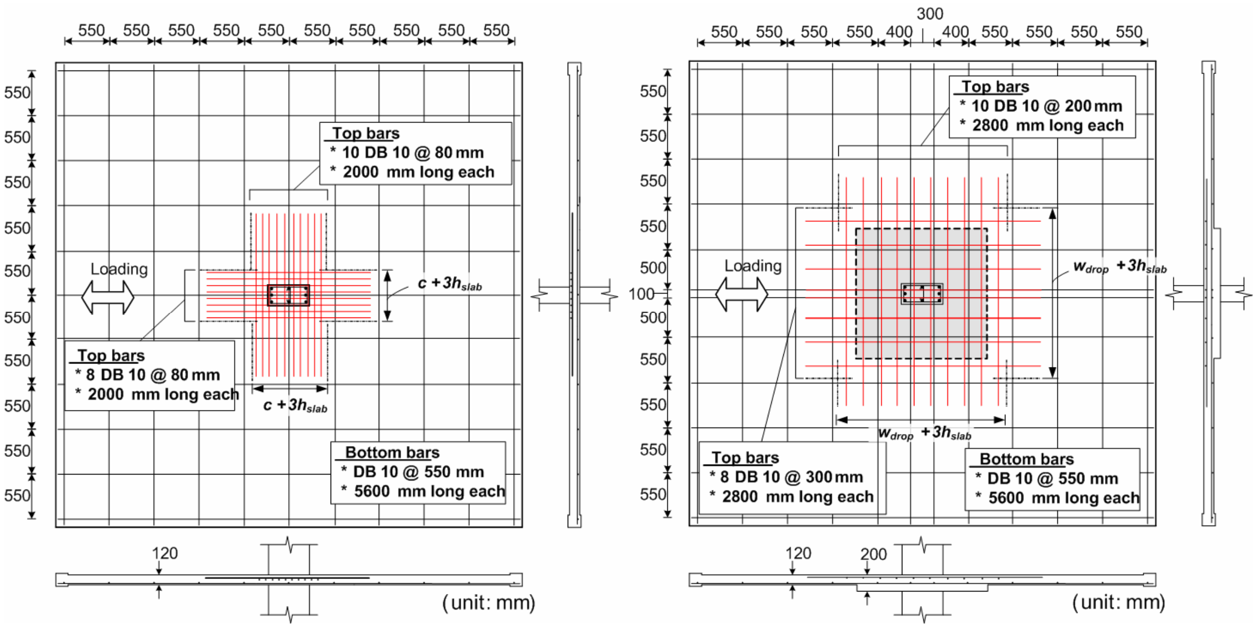
Figure 1 Interior slab–column connection specimen with drop panel and its dimensions

In both specimens, all PT tendons were grade 270, seven–wire strands with nominal diameter of 12.7 mm. Eight straight tendons were banded in the direction of loading with a spacing of 350 mm, except the two tendons located near the column were spaced at 290 mm interval. Other eight straight tendons were uniformly distributed in the orthogonal direction with spacing of 700 mm. Each tendon was inserted into a galvanized duct. After considering that the concrete slab gained sufficient strength, each tendon was tensioned individually with a hydraulic jack. The average applied force in each tendon was approximately 147.15 kN, corresponding to the stress of 0.80 f_{pu}. After prestressing the tendons and filling the end recesses, all galvanized ducts were grouted to provide an effective bond between the tendons and the ducts. The tendon layouts of both specimens are depicted in Fig. 2.



(a) specimen without drop panel (b) specimen with drop panel

Figure 2 Layout of prestressing strands:



(a) specimen without drop pane (b) specimen with drop panel

Figure 3 Bonded reinforcing bars in slabs

The top slab bars in the specimen without drop panel were concentrated only at the connection region with a spacing of 80 mm, as shown in Fig. 3a. These bars were cut off at a distance of 1000 mm from the column center. On the other hand, the top slab bars in the specimen with drop panel were placed uniformly over the drop panel in direction of loading with a spacing of 300 mm, except the two bars over the column were spaced at 100 mm interval. In the orthogonal direction of loading, top bars with equal spacing of 200 mm were provided. All top bars in the specimen with drop panel, as shown in Fig. 3b, were cut off at a distance of 1400 mm from the column center.

Fig. 3 also shows the layout of bottom slab bars of both specimens. The bottom bars in both specimens were spaced at 550 mm intervals throughout the slab. A continuous bottom bar through the column core was provided in each direction in the specimen without drop panel (see Fig. 3a). On the other hand, two continuous bottom bars were placed over the column in each direction in the specimen with drop panel (see Fig. 3b) to satisfy the ACI 318–05's requirements to prevent progressive collapse in the event of a connection shear failure (Section 13.3.8.5 of ACI 318–05).

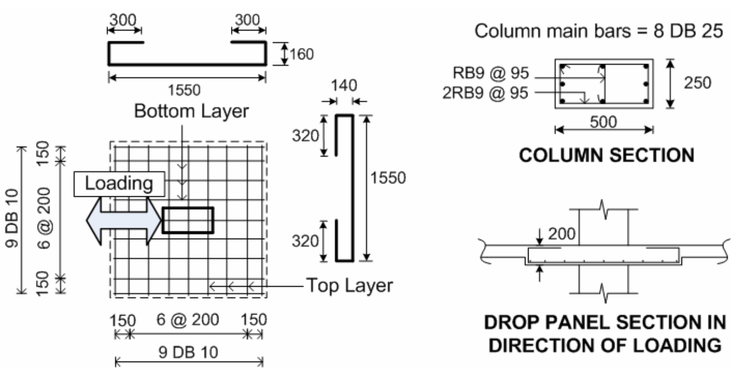


Figure 4 Bonded reinforcing bars in drop panel and column of the specimen with drop panel

For the specimen with drop panel, nine bottom bars of DB10 were placed over the drop panel in loading direction. Other nine bottom bars of DB10 were also provided within the drop panel in the orthogonal direction. The development length of all drop panel bars was about 40 times bar diameter (400 mm). For the column in both specimens, sufficient transverse and longitudinal reinforcing bars were provided so that the column would remain elastic without experiencing shear failure during the test. The layout of conventional reinforcing bars in drop panel and column zones of the specimen with drop panel is depicted in Fig. 4. The material properties of both specimens are presented in Table 1. For complete details on the design and construction of these two specimens, refer elsewhere (Warnitchai et al. 2004, Tandian 2006).

Table 1 Material properties

Specimen	Concrete Slab at Test Day f_c'	Reinforcing Bars		Prestressing Strands	
		Yield Strength	Tensile Strength	Yield Strength	Tensile Strength
Specimen without drop panel	41MPa	503MPa	578MPa	1,780MPa	1,902MPa
Specimen with drop panel	46MPa	324MPa	491MPa	1,763MPa	1,947MPa

2.2.Testing of Specimens

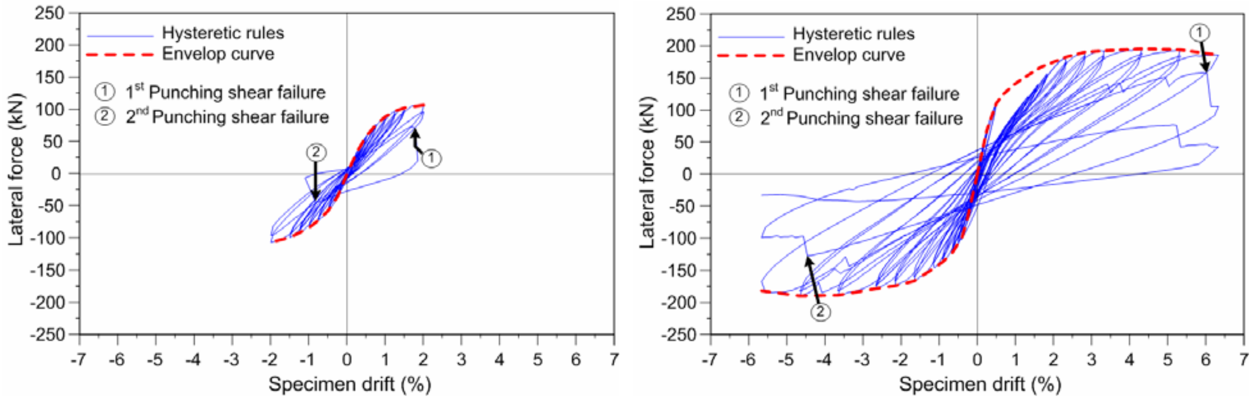
It is well known that a major parameter that influences the lateral displacement capacity of slab–column connections is the gravity shear ratio (V_g/V_0), where V_g is the direct gravity shear force acting on the slab critical section and V_0 is the slab punching strength in the absence of moment transfer as defined in ACI 318-05. The lateral displacement capacity and ductility generally drops as the magnitude of the connection gravity shear ratio (V_g/V_0) increases (Pan and Moehle 1989, Kang and Wallace 2006). Hence, the specimen without drop panel was loaded by a number of sand bags on top and below the slab to simulate the gravity shear ratio (V_g/V_0) of 0.28, which is approximately the average value of those found in slab-column frame buildings in Thailand (Warnitchai et al. 2004). The same gravity loading was also applied to the specimen with drop panel. But since the slab in the connection region was thicker due to the presence of drop panel, the gravity shear ratio of 0.13 was obtained for this case.

As depicted in Fig. 1, the lateral load was applied to both specimens by a MTS servo controlled hydraulic actuator attached to the top of column. The hydraulic actuator was mounted to a rigid reaction wall. A typical displacement–controlled reversed cyclic lateral loading test was carried out to both specimens with monotonically increasing target drifts of 0.25%, 0.50%, 0.75%, 1.00%, 1.25%, 1.50%, 2.00%, 2.50%, 3.00%, 4.00%, and so on… At each target drift, two complete cyclic displacement loops were conducted. The loading was terminated after the punching cone had formed completely. Note that the respective target drift is defined as the ratio of the lateral displacement of column at lateral loading point to the column height, which is 1.8 meter.

During the testing of both specimens, all measurement data were recorded at each loading step. The data measured in the experimental program included: (1) lateral force and displacement at the top column end, (2) lateral displacement and rigid–body twisting angle of slab, (3) bending curvature of slab in front of and behind the column, and (4) strain profile in reinforcing bars and prestressing strands. Lateral force was measured by a force sensor connected to the hydraulic actuator. Lateral displacements were determined from lateral displacement readings taken at the top and bottom ends of column. Further details on loading and instrumentation of the specimen without drop panel and the specimen with drop panel are reported in Pongpornsup (2003) and Tandian (2006), respectively.

3. EXPERIMENTAL RESULTS

Due to space limitation, only some results are presented in this paper. The relationship between lateral force and drift is presented in Fig. 5a and 5b. Fig. 5a shows the force–drift relationship of the specimen without drop panel, while Fig. 5b depicts that of the specimen with drop panel. From both figures, the hysteretic loop of both specimens in every loading cycle was obviously long and narrow before punching occurred, demonstrating a limited ability to dissipate energy. Each specimen behaved like a linear elastic system with viscous damping. The specimen stiffness decreasingly degraded as the drift level became higher. No pinching phenomenon was observed in the hysteretic loops for both specimens.



(a) Specimen without drop pane

(b) Specimen with drop panel

Figure 5 Lateral force-drift results of both specimens

As shown in Fig. 5a and 5b, compared to the specimen without drop panel, the specimen with drop panel notably possessed drift capacity about three times as much prior to punching. The specimen with drop panel was able to reach 6.0% drift. On the other hand, the one without drop panel could only undergo 2.0% drift. The results seem to indicate that the gravity shear ratio plays an important role in determining the drift capacity at punching failure. In addition, the specimen with drop panel apparently failed in a more ductile manner than the one without drop panel. The specimen with drop panel experienced a saturation of peak load for the drift 2.5% to 6.0%, indicating flexural yielding took place before punching failure. On the other hand, the specimen without drop panel suddenly failed in punching shear while no peak load saturation was perceived in advance. The results in Fig. 5a and 5b also show that the presence of drop panel resulted in a significant increase in the strength of the slab-column connection.

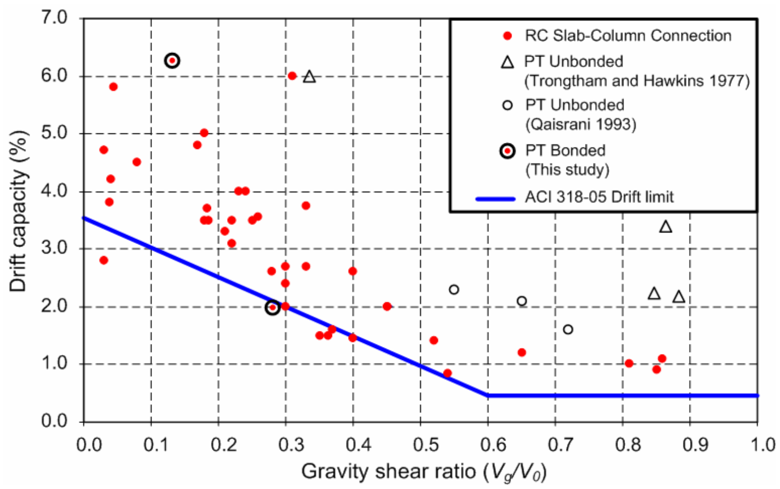


Figure 6 Gravity shear ratio versus drift capacity at punching for RC and PT slab-column connections

Figure 6 shows a plot of the gravity shear ratio and drift capacity at punching of both specimens, along with other test results of slab-column specimens without shear reinforcement. Most of the test results of

RC slab-column specimens were collected and compiled by Pan and Moehle (1989), while those of unbonded PT slab-column interior connections were tested by Trongtham and Hawkins (1977) and Qaisrani (1993) (and summarized by Kang and Wallace (2006)). The ACI 318-05's design drift limit for slab-column connections is also plotted in Fig. 6 for reference.

Contrary to the results of unbonded PT slab-column connections, where the drift capacity is roughly twice the ACI 318-05's drift limit or more in all cases, the results of the two bonded PT slab-column connections does not show the same trend. Instead, the test results of both specimens are more or less consistent with those of RC specimens, suggesting that the ACI 318-05's design drift limit could be used for bonded PT slab-column connections as well. Nevertheless, since the number of test data for bonded PT connections is very limited, more test data are definitely required before a clear conclusion can be drawn on this issue.

4. SUMMARY AND CONCLUSIONS

Two three–fifth scaled non–seismically designed bonded post–tensioned (PT) interior slab–column connections were subjected to similar gravity loading as well as quasi–static reversed cyclic lateral loading pattern. The first specimen was without drop panel. The second specimen was with drop panel. Based on the experimental results of both specimens, the following conclusions are drawn:

1. During the test, each specimen behaved like a linear elastic system with low energy dissipation, as indicated by long and narrow hysteretic loops. No pinching phenomenon was observed in the hysteretic loops of both specimens.

2. The specimen with drop panel apparently failed in more ductile manner than the one without drop panel. The specimen with drop panel experienced saturation of peak load from about 2.5% to 6.0% drifts, indicating flexural yielding took place before punching failure. The specimen with drop panel was able to reach 6.0% drift, whereas the one without drop panel could only undergo 2.0% drift. Nevertheless, punching failure still persisted in both specimens, with and without drop panel.

3. The test results suggest that the gravity shear ratio (V_g/V_0) is the major variable which governs the drift capacity and ductility of bonded PT interior connections, as comparable to both reinforced concrete and unbonded PT flat plate connections.

4. The test results also suggest that the ACI 318-05's design drift limit could be used for bonded PT slab-column connections. More testing on bonded PT connections is recommended to confirm this point.

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