# Seismic performance enhancement of post–tensioned flat plate systems with drop panel

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**ABSTRACT:**

The seismic performance of a three–fifth scaled bonded post–tensioned (PT) interior slab–column connection model with drop panel was experimentally assessed. The model was intended to represent an improved design of typical slab–column connections without drop panel in Thailand. A quasi–static reversed cyclic loading test was carried out on the model up to failure. The effect of incorporating drop panel in the PT slab–column region on its seismic performance was identified by comparing the test results with those of a connection model without drop panel, which has been tested earlier. The experimental results show that the connection with drop panel was able to undergo up to 6% drift prior to punching shear. Compared to the connection without drop panel, the connection with drop panel has remarkably demonstrated drift capacity approximately thrice as much. Furthermore, the connection with drop panel considerably showed more ductility and energy dissipation capacity than the one without drop panel.

## 1 INTRODUCTION

Post–tensioned (PT) flat plate construction has long been popular in Thailand for medium–rise to high–rise buildings. As a general design practice, flat plate structures are usually designed to carry only gravity load, while concrete shear walls are assumed to resist lateral wind load. The flat plate structures are 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 caused by transfer mechanism of earthquake–induced unbalanced moment between slab and column. Such punching 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, Martinez–Cruzado et al. 1994, Warnitchai et al. 2004, Kang and Wallace 2006, Gayed and Ghali 2006). Almost all tested PT specimens were hitherto made to represent unbonded flat plate connections. Only one PT specimen was tested (Warnitchai et al. 2004) to assess the seismic behavior of bonded flat plate connections, which are the prevailing type of flat plate construction in Thailand.

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 the 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 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 in the specimen with 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 so 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, if any, 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 Figure 1.

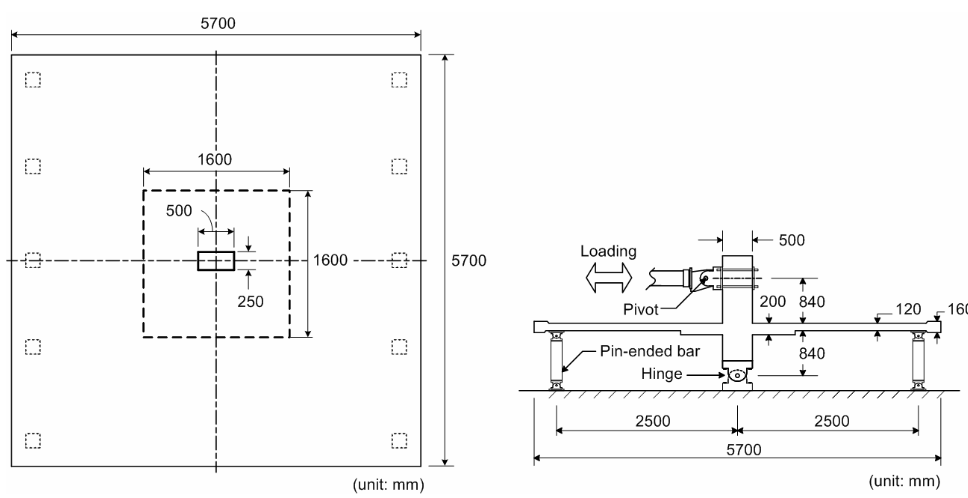



Figure 1 Interior slab–column connection specimen with drop panel and its dimensions

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.

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 stress 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 Figure 2.

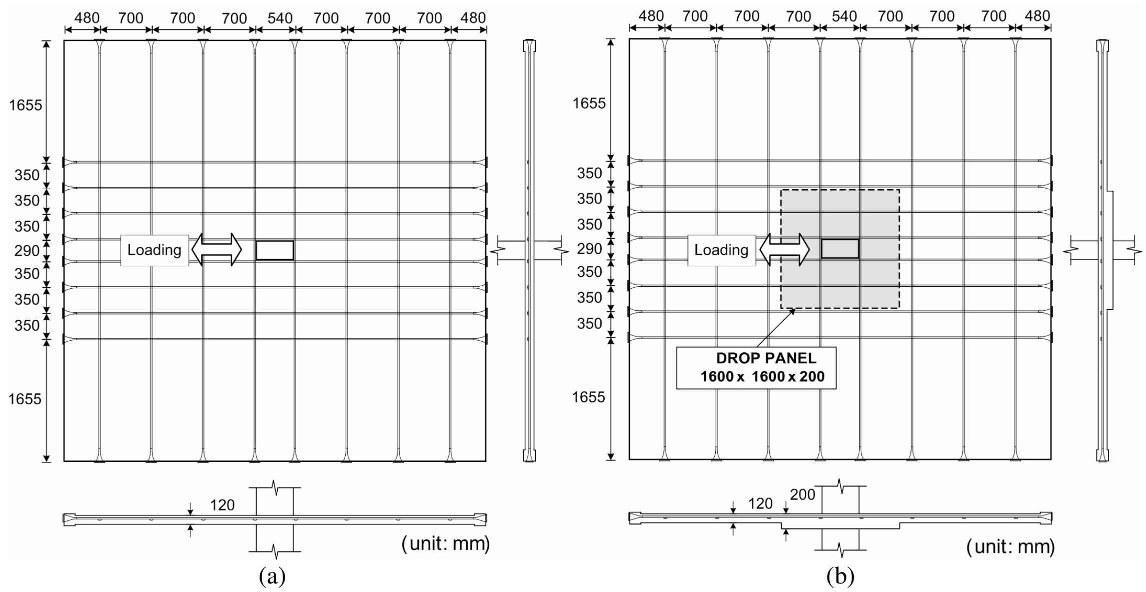
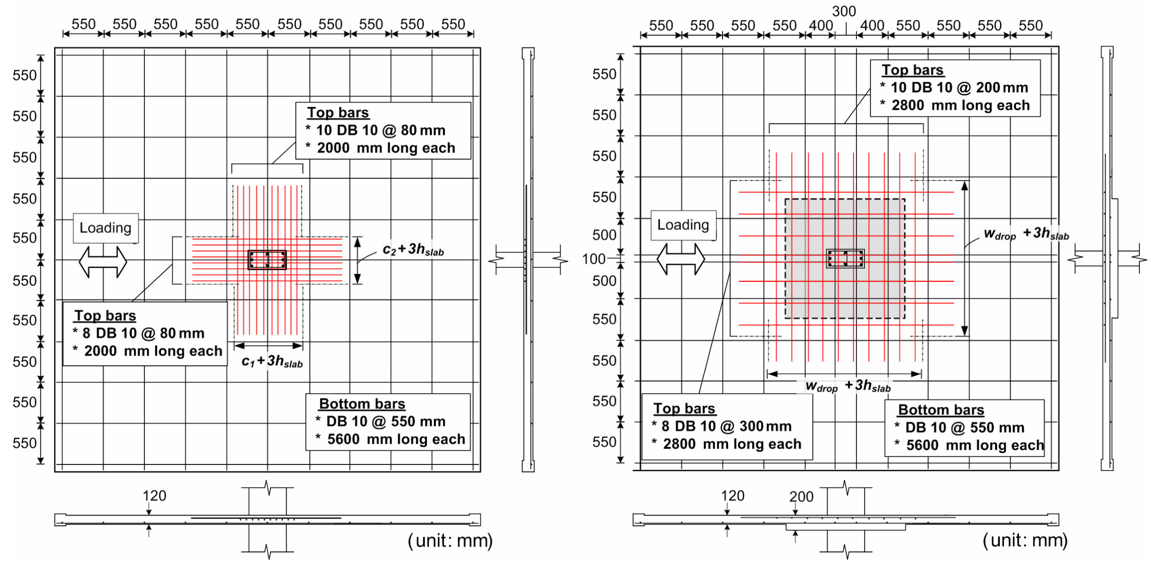



Figure 2 Layout of prestressing strands: (a) specimen without drop panel, (b) specimen with drop panel

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 Figure 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 Figure 3b, were cut off at a distance of 1400 mm from the column center.

Figure 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 requirements to prevent progressive collapse in the event of a connection shear failure.




(a) (b)

Figure 3 Bonded reinforcing bars in slabs: (a) specimen without drop panel, (b) 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 Figure 4. The material properties of both specimens are presented in Table 1. For complete details on the design and construction of both specimens, refer elsewhere (Warnitchai et al. 2004, Tandian 2006).

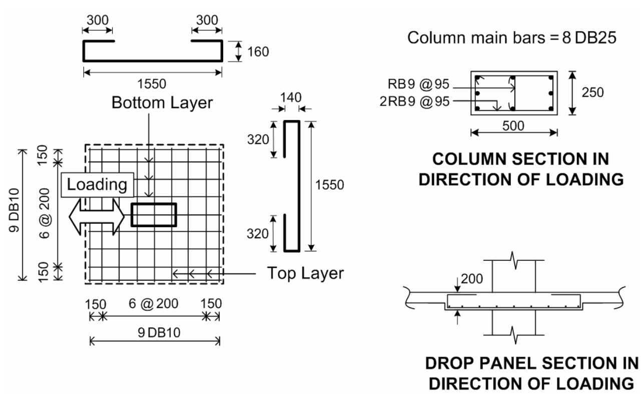



Figure 4 Bonded reinforcing bars in drop panel and column of the specimen with drop panel

**Table 1 Material properties**

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

2.2 **Testing of Specimens**

In this experimental program, the gravity loading applied to both specimens simulated the dead load and likely live load (ATC, 1996) of the average post–tensioned (PT) slab–column connections in Thailand. All specimens were subjected to similar gravity loading, so that similar magnitude of direct gravity shear force ($V_g$) in the column vicinity of the connections could be maintained. Thus, all slabs were subjected to the combination of slab self–weight and sand bags (50 kg each) with appropriate amount and location. The quantity and location of sand bags in the test slabs were determined from elastic finite element analysis.

As depicted in Figure 5, lateral load was applied to all specimens by a MTS servo controlled hydraulic actuator attached to the top of column. The hydraulic actuator was mounted to a rigid reaction wall. The North–South direction was designated as the loading direction and the East–West direction as the transverse direction. As shown in Figure 6, a typical displacement–controlled reversed cyclic lateral loading test was carried out to both specimens with monotonically increasing target drifts of $\pm 0.25\%$, $\pm 0.50\%$, $\pm 0.75\%$, $\pm 1.00\%$, $\pm 1.25\%$, $\pm 1.50\%$, $\pm 2.00\%$, ... At each target drift, two complete cyclic displacement loops were made. The loading was stopped as 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 (See Fig. 1).

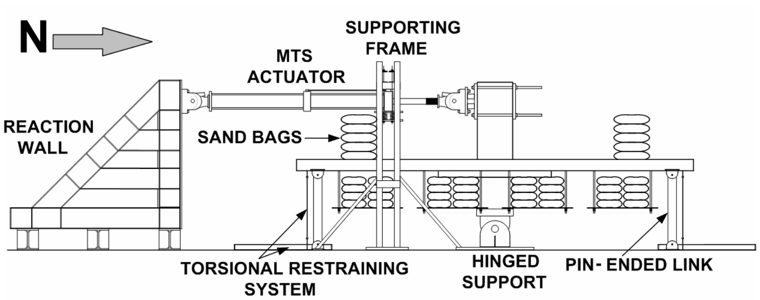



Figure 5 Experimental setup

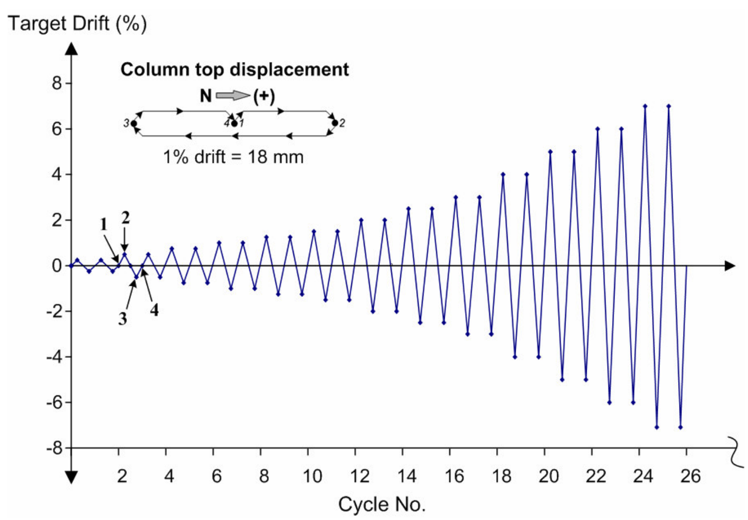



Figure 6 Pattern of lateral loading

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

The relation between lateral force and drift is presented in Figures 7 and 8. Figure 7 shows the force–drift relationship of the specimen without drop panel (Warnitchai et al. 2004), while Figure 8 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 as a linear elastic system with existent viscous damping. The specimen stiffness degraded as the drift level went higher. No pinching was observed in the hysteretic loops of both specimens.

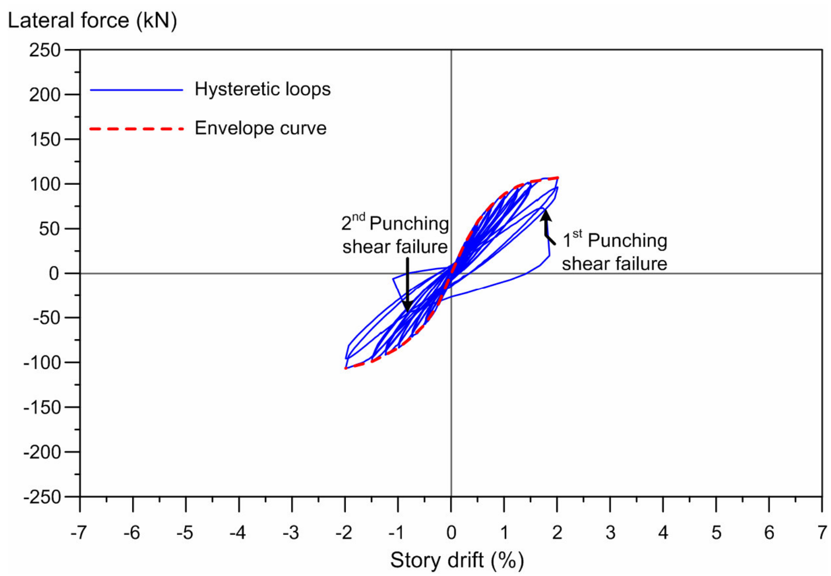



Figure 7 Lateral force–drift relationships for the specimen without drop panel

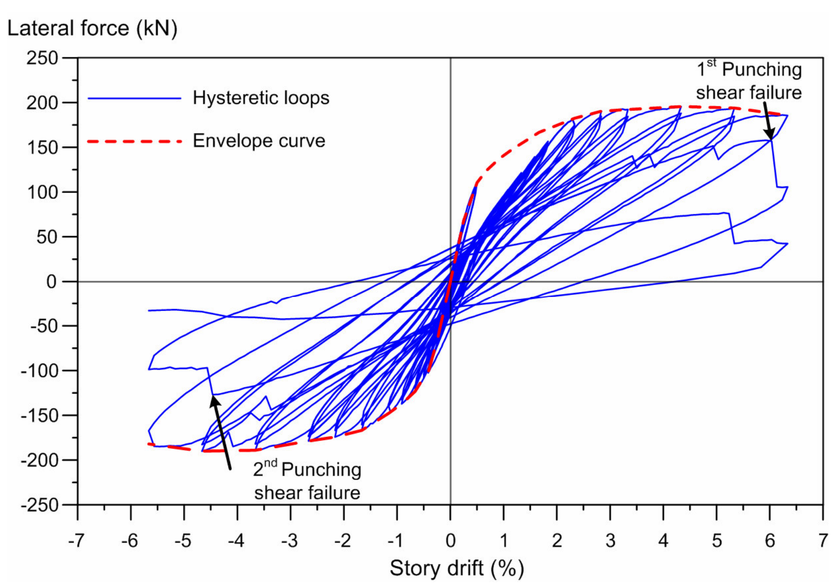



Figure 8 Lateral force–drift relationships for the specimen with drop panel

As shown in Figures 7 and 8, compared to the specimen without drop panel, the specimen with drop panel notably possessed drift capacity about thrice as much prior to punching. The specimen with drop panel was able to reach 6.0% drift, whereas the one without drop panel could only undergo 2.0% drift. In addition, 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. Meanwhile, the specimen without drop panel suddenly failed in punching shear while no peak load saturation was perceived in advance.

The gravity shear ratio ($V_g/V_0$) of either reinforced concrete (RC) or unbonded post–tensioned (PT) slab–column connections was found to significantly affect the drift capacity and the ductility of the connections at punching. As defined in ACI 318–05 (ACI, 2005), $V_g$ is the gravity shear force acting on the slab critical section, while $V_0$ is the slab punching strength in the absence of moment transfer. As reviewed by Pan & Moehle (1989) and Kang & Wallace (2006), the connection drift capacity and ductility drops as the magnitude of the connection gravity shear ratio increases. This study confirms the applicability of this finding to bonded PT slab–column connections. As discussed before, the specimen with drop panel ($V_g/V_0$=0.13) showed more drift capacity and ductility than the one without drop panel, which had higher gravity shear ratio ($V_g/V_0$=0.28). The gravity shear ratio of each specimen was computed using elastic finite element analysis.

Energy dissipation capacity is an important parameter for evaluating the structure capacity to survive in cyclic loading without collapse. Figure 9 showed the cumulative dissipated energy of all specimens prior to punching. The dissipated energy within loop or cycle $i$ ($E_{Di}$) was obtained from the area enclosed by the force–displacement curve within loop or cycle $i$. The cumulative dissipated energy up to $j$ percent drift is defined as the summation of the dissipated energy of all cycles which the specimen experienced up to $j$ percent drift. Those cycles that resulted in a drop in lateral load resistance of more than 20% of the peak load were excluded in the calculation. From Figure 9, the specimen with drop panel exhibited the ability to dissipate energy larger than the specimen without drop panel by almost 650%. In advance of the punching shear occurrence, the specimen with drop panel was able to dissipate energy up to 60.37 MN.mm, while the one without drop panel could only dissipate energy up to 8.09 MN.mm.

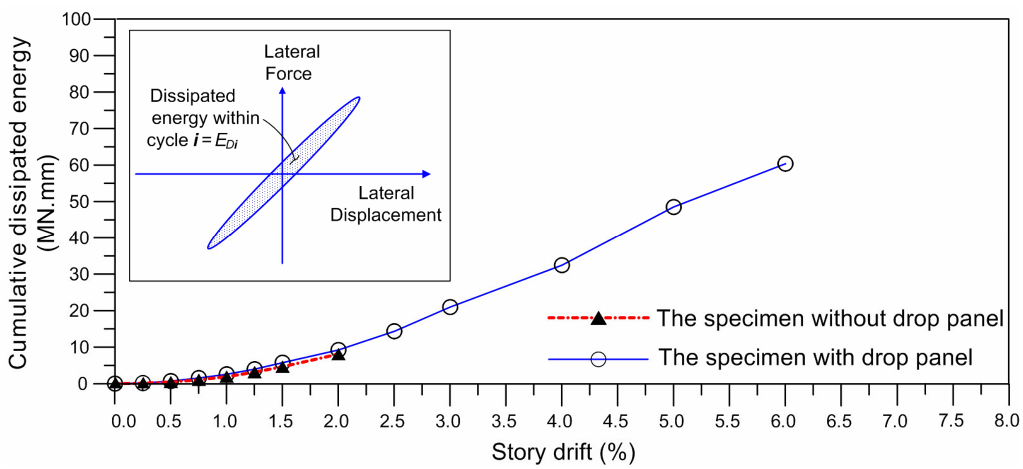



Figure 9 Cumulative dissipated energy of both specimens, without and with drop panel

## 4 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, tested earlier by Warnitchai et al. (2004), was without drop panel. The second specimen, tested in this study, 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 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.

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