SEISMIC PERFORMANCE ENHANCEMENT OF POST–TENSIONED FLAT PLATE SYSTEMS WITH SHEAR REINFORCEMENTS

U. Prawatwong[1], P. Warnitchai[2], A. Luckkikanun[3]

[1] *Lecturer, Suranaree University of Technology, unnop@sut.ac.th*
[2] *Professor, Asian Institute of Technology, pennung@ait.ac.th*
[3] *Master student, Suranaree University of Technology, luckkikanun.a@gmail.com*

Abstract

The use of post-tensioned (PT) slabs for building structural systems has become increasingly popular in many countries, but little research has been conducted on the seismic performance of bonded PT slab-column connections. It is widely known that slab-column connections are the most critical regions in a flat plate system. Under a strong earthquake ground motion, sudden and brittle punching failure may occur at a slab-column connection region due to a combination of direct gravity shear and eccentric shear from an excessive earthquake-induced unbalanced moment between slab and column. In addition, extensive cracks in the connection region caused by repeated reversals of large lateral deformation may significantly deteriorate the shear strength of the connection. The punching shear failure at one connection may, in turn, initiate a progressive collapse of the entire building structures as notoriously shown in some literature.

There are several solutions to the problem of punching failure in slabs near the connections. The common solutions used in practice in Thailand are the use of drop panels or slab shear reinforcements. In an attempt to eliminate the use of drop panels, the use of flat plate floor systems consisting of a PT concrete slab-column system incorporating shear reinforcements within the slab-column connection region has become increasingly popular in medium to high-rise buildings in Thailand. However, no experimental studies of bonded PT slab-column connections involving shear reinforcements subjected to earthquake-type loading have been found in any literature. Very few guidelines and little information are available for designers to design the connections under earthquake loading. Therefore, experimental investigations on the seismic performance of bonded PT slab-column connections with shear reinforcements are needed.

In this paper, the results of a series of tests on two 3/5 scaled bonded PT interior slab–column connection models under simulated-earthquake loading will be presented. The purpose of the tests is to investigate the seismic performance of bonded PT interior slab-column connections containing shear reinforcements. In the first model, the slab-column connection was reinforced with shear reinforcements in the form of closed-hoop stirrups usually found in Thailand. In the second model, the slab-column connection was reinforced against punching shear by type 2 double-head studs according to ASTM A1044M. Both models were tested under a constant gravity load level combined with incrementally increasing lateral displacement reversals up to failure. During the tests, the models were carefully instrumented to provide detailed data on its behavior throughout its entire loading history. Relevant design equations suggested by ACI 318-08 Building Code provisions as well as previous similar tests by others were compared with the test results from this study. The test results suggested that the shear reinforcement in the form of double-head studs effectively and significantly enhances the poor performance of the typical bonded PT interior connections. However, the experimental results from this study pointed out that the conventional shear reinforcement in the form of closed-hoop stirrups may not provide a significant increase in punching shear strength for the thin slab under earthquake type loading.

Keywords: post-tensioned slab; slab-column connection; punching shear; shear reinforcement; double head stud

1. Introduction

The post-tensioned (PT) flat plate is a simple structural system that consists of a PT flat slab support directly by columns. This system is very popular as a gravity load-resisting system for slab-column frames in many countries, primarily due to its ease of construction and architectural and serviceability reasons. The long development of post-tensioning systems for cast-in-place flat plate in each country has resulted in either an "unbonded" system or a "bonded" system. Bonded systems are more popular in Thailand and Australia because the practical benefit is that, the bond between the concrete and the tendons offers more flexibility regarding structural modifications such as openings for stairwells, utility access, and future expansion. It is widely known that slab-column connections are the most critical regions in a flat plate system. Under a strong earthquake ground motion, sudden and brittle punching failure may occur at a slab-column connection region due to a combination of direct gravity shear and eccentric shear from an excessive earthquake-induced unbalanced moment between slab and column. In addition, extensive cracks in the connection region caused by repeated reversals of large lateral deformation may significantly deteriorate the shear strength of the connection. The punching shear failure at one connection may, in turn, initiate a progressive collapse of the entire building structures as notoriously shown in some literature [1].

Although extensive tests on the seismic performance of slab-column connections have been carried out over the past four decades, most of these works focused on the seismic response of reinforced concrete (RC) flat plates. A limited number of studies [2, 3, 4, 5, 6] investigated the seismic capacity of PT flat plates. The updated database of slab-column connection tests in literature was collected and reviewed in [7]. As shown in the database, almost all tested PT specimens were hitherto made to represent unbonded flat plate connections. Only two PT specimens were tested [6] to assess the seismic behavior of bonded flat plate connections, which are the prevailing type of flat plate construction in Thailand.

To prevent slab-column connections from punching failure, there are several solutions used in practice. A common solution is to increase the slab thickness around the columns; this can be achieved by the use of drop panels. Under earthquake loading, the test results in [6] suggested that a properly designed drop panel is an effective way to greatly enhance the overall performance of the bonded PT slab-column connection. However, it should be noted that this solution required additional concrete and labor-intensive formwork. In an attempt to eliminate the use of drop panels, the use of flat plate floor systems consisting of a PT slab incorporating shear reinforcements within the slab-column connection region as recommended in [8] has become increasingly popular in medium to high-rise buildings in Thailand. However, no experimental results of bonded PT slab-column connections involving shear reinforcements subjected to earthquake-type loading have been found in any literature. The seismic behaviors of bonded PT slab-column connections with shear reinforcements are of severe lack. Very few guidelines and little information are available for designers to design the connections under earthquake loading. As a result, the effect of shear reinforcements on deformation capacity enhancement for bonded PT slab-column connections under earthquake-type loading is still questionable. Therefore, experimental investigations on the seismic behavior of bonded PT slab-column connections with shear reinforcements are necessary.

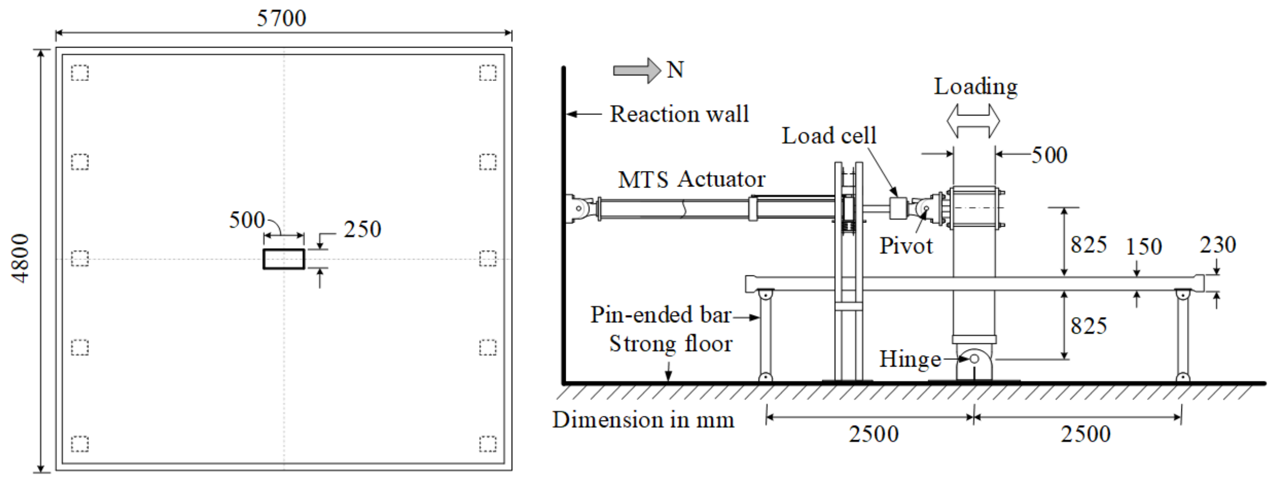
This paper deals with reversed-cyclic tests to failure on two three-fifth scale models of bonded PT interior slab-column connections with shear reinforcements in the form of closed-hoop stirrups and double-headed studs. Each specimen was subjected to a lateral quasi-static cyclic loading routine to investigate its seismic performance through the elastic and inelastic ranges and finally until failure. The effect of incorporating shear reinforcements in the PT slab-column region on its seismic performance was identified by comparing the test results with those of connection models without shear reinforcement, which has been tested earlier as reported in [6]. The results from this study and the comparisons will provide useful information on the cyclic performance of bonded PT interior slab-column connections with shear reinforcements. It will be a guideline for structural designers in the future.

2. Experimental program

2.1. Specimens description

Two specimens with shear reinforcements were designed and constructed after typical connections found in most PT flat plate buildings in Thailand. The typical span, story height, and slab thickness of flat plate with shear reinforcement building in Thailand were in the range of 5.70 to 10.70 meters, 3.00 meters and 0.25 meters, respectively. Both specimens were approximately 3/5 scaled of the typical interior slab-column connections in the prototype buildings. The first specimen, denoted by SS1, was designed to investigate the seismic performance of bonded PT slab-column connections containing punching shear reinforcement in the form of conventional closed-hoop stirrups as found in most PT flat plate buildings in Thailand. The second specimen, denoted by SS2, was designed to investigate the effect of double-head studs to improve the seismic performance of the first connection. Each of the specimens was identical in slab dimension, column dimension, tendon layout, and prestressing forces. All of them were of normal weight concrete.

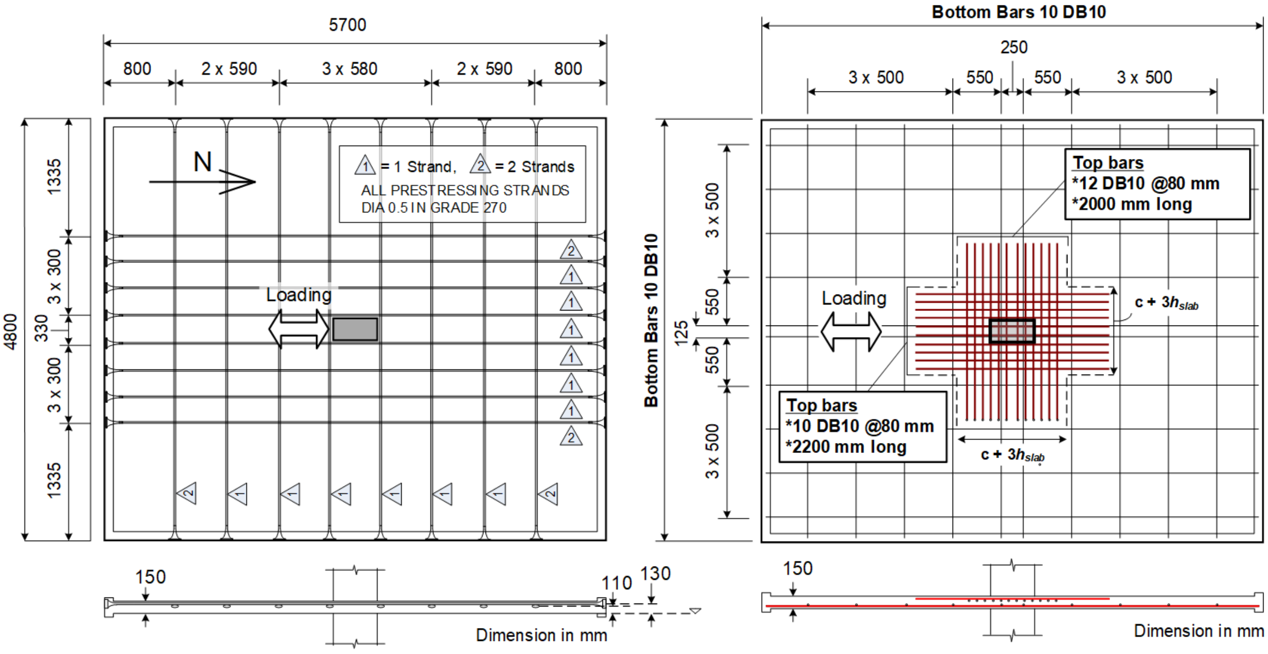
Fig. 1 shows the dimensions of the tested specimens. The slabs were all 5000–mm spans, one of which was reinforced against punching shear with shear reinforcements in the form of closed-hoop stirrups and the other was reinforced by type 2 double-head studs according to ASTM standard [9]. The thickness of the slab in each of the specimens was 150 mm. The size of the column was 250 x 500 mm, while the height was 1800 mm. As each specimen was developed based on the assumption that inflection points in the interior connection under earthquake-type loading occur at slab mid-span and column mid-story, half the total height of an interior column above and below the slab and half of the slab spans between adjacent columns on all four sides were modeled. To simulate a moment-free boundary condition, pin connections were attached to the points of contra-flexure under lateral loading. This model of connection was designed to produce bending moment and shear of the slab comparable to the prototype in the vicinity of the column where the most damage was expected. The validation of this model and assumption were well explained in [10].



(a) Plan view (b) Elevation

Fig. 1 - Interior slab column connection specimens and its dimensions

Fig. 2 and Fig. 3 provide the details of reinforcement in both specimens. In both specimens, all strands in PT slab were ASTM A-416, Grade 270, 7–wire strands with nominal diameter of 12.7 mm. Eight straight tendons with ten strands were banded in the direction of loading with a spacing of 300 mm, except the two strands located near to the column had spacing of 330 mm. The other eight straight tendons with ten strands were distributed uniformly in perpendicular to the loading direction. Each strand was inserted into a flat (20 mm in height) galvanized ducts. To prevent damage due to high concentrations of stresses at the edges of the slab, an edge beam with sufficient reinforcing bars was provided on all sides of the slab. After the concrete slab gained sufficient strength, each strand was tensioned individually by a hydraulic jack. The average applied stress in each strand was approximately 80% of ultimate strength. After prestressing the strands and filling the end recesses, all galvanized ducts were grouted to provide an effective bond between the strands and the ducts. The tendons layouts and their profile in the slab of the specimens are shown in Fig. 2(a).

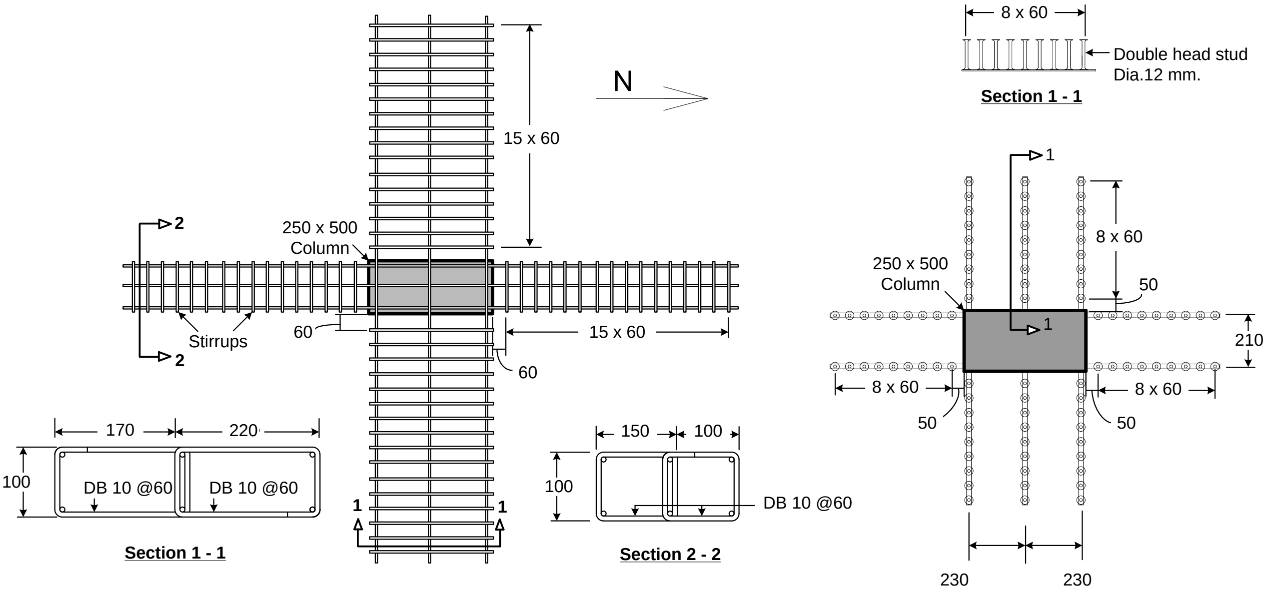


(a) Layout of prestressing strands (b) Top and bottom bars

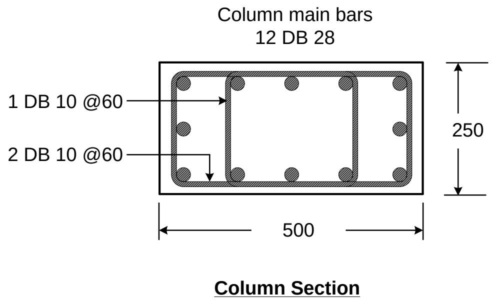
Fig. 2 - Layout of prestressing strands, and supplementary reinforcements in slabs of both specimens

Fig. 2(b) shows the supplementary reinforcement bars in the slabs of both specimens. In the slabs, DB10 (10 mm diameter) deformed bars were used for the supplementary top and bottom reinforcements. Both specimens contained the top reinforcement bars at the top of its slab according to ACI 318-14 Code [11] Section 8.6.2.3. For prestressed slabs, the code requires that a minimum area of bonded deformed longitudinal reinforcement equal to 0.00075 hl, where h is the total slab thickness and l is length of span in direction parallel to that of the reinforcement being determined, should be provided in the pre-compressed tension zone over the effective width of the slab near the supporting column in both directions. The top reinforcement bars were distributed in each direction within an effective width of $c + 3h$ and extend away from the column face at least $l_n/6$, where c is the column width and l_n is the length of clear span, in accordance with ACI 318-14 Code Section 8.7.5.3 and 8.7.5.5.1. For bottom reinforcement, DB10 deformed bars were provided as temperature and shrinkage reinforcement in both directions. A nominal clear concrete cover of 10 mm was specified for both top and bottom reinforcement. All bar arrangements were in such a way that the top and bottom bars in the direction of loading were placed at the outmost layer.

Fig. 3 shows the details of shear reinforcements in PT slabs near the slab-column connection of each of the test specimens and column reinforcement. Fig. 3(a) shows the layout of conventional stirrups reinforcement in Specimen SS1. As shown in section 1-1 and 2-2 of Fig. 3(a), each stirrup is DB10 bends in a closed-hoop stirrup. The spacing is 60 mm extends from column face 960 mm each direction. On the other hand, Fig. 3(b) shows the stud-shear reinforcement layout in Specimen SS2. Ten stud rails were placed around the column. Stud spacing is 60 mm (0.5d). The first studs were placed away from the column face 50 mm. As mentioned earlier, all of the studs are type 2 double-head stud following ASTM standard [9]. The total height of the stud rails is 120 mm. The details design of the stud rails can be seen in [12] Fig. 3(a) also provides column reinforcement details of both specimens. Twelve bars DB28 yielding stresses of 400 MPa were the main reinforcement in the column. The shear reinforcements of the column were stirrups fabricated from bars type DB10 with a spacing of 60 mm. The nominal clear cover for the column reinforcement is 15 mm. It is expected that the column could behave in elastic manner during the test.



(a) Conventional shear stirrup reinforcement in SS1 (b) Stud-shear reinforcement in SS2



(c) Column reinforcement in both specimens

Fig. 3 - Details of shear reinforcement in PT slabs and column reinforcement of the test specimens

To assess the actual strength of the concrete used in each of the test specimens, compression tests on cylinders of 150 x 300 mm were carried out on the testing day. The results are listed in Table 1. The yielding stress of DB10, 7-wires strand, and double-head stud used in the models are also presented in Table 1.

Table 1. Material properties

Concrete	**Compressive strength on test day, MPa**		**Steel**	**Yielding stress, MPa**	
	SS1	**SS2**		**SS1**	**SS2**
Bottom column	43.26 (94 days)	47.41 (138 days)	DB10	416	374
Top column	61.91 (35 days)	49.45 (31 days)	7-wires strand	1,729	1,710
Slab	52.02 (38 days)	38.52 (33 Days)	Shear stud	-	380

2.2. Testing of specimens

It is well known that a major parameter that influences the lateral displacement capacity of the 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. In this study, 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 sandbags with the appropriate amount and location. The quantity and location of sandbags in the test slabs were determined from elastic finite element analysis such that the computed gravity shear ratio (V_g/V_0) was equal to 0.28, which is the same as Specimen S1 without shear reinforcement from the previous test as reported in [6].

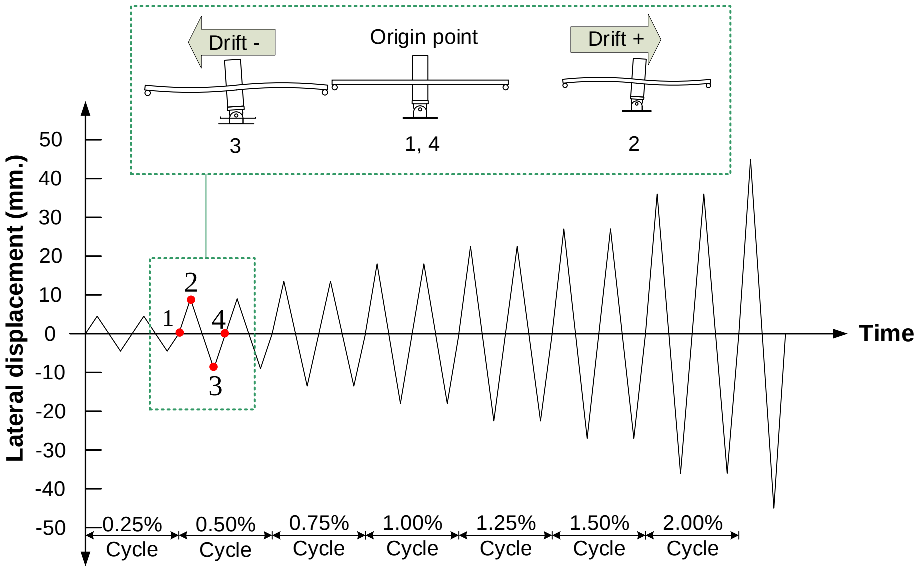


Fig. 4 - Pattern of lateral loading

As depicted in Fig. 1, the lateral load was applied to each of the specimens by an MTS servo-controlled hydraulic actuator attached to the top of the column. The hydraulic actuator was mounted to a rigid reaction wall after the application of the sandbags. The North-South direction was designated as the loading direction and the East-West direction as the transverse direction. 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. Fig. 4 depicts the pattern of the lateral cyclic loading. 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 the column at lateral loading point to the column height, which is 1.8 meter.

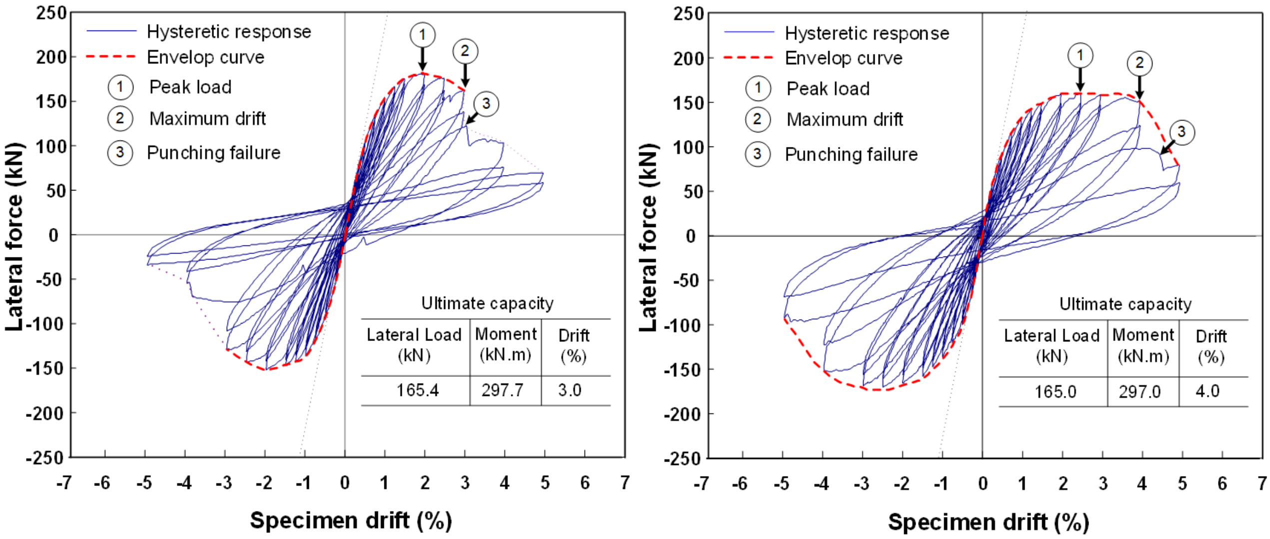
During the tests, all models were carefully instrumented to provide detailed data on its behavior throughout its entire loading history. The data measured and recorded include: (1) lateral force and displacement at the top column, (2) lateral displacement and rigid–body twisting angle of slab, (3) strain profile in reinforcing bars and prestressing strands, and (4) strain in punching shear reinforcements. In addition, photos were taken and at peak positive and negative drift every cycle of loading to record the development of visible cracks on the top and bottom slab surface. Five video cameras were recorded continuously throughout the tests.

3. Experimental results

Experimental results obtained from tests of both specimens are presented and discussed in the following sections. Due to space limitation, only some results are presented and discussed in this paper. The discussion includes the seismic performance of SS1 and SS2, which highlight the overall behaviors of the test connections under the lateral cyclic loading applied in this study. Subsequently, further analyses of the lateral force-drift response in terms of eccentric shear stresses and drift capacity are discussed. Comparisons with previous similar tests by others are made.

3.1 Overall response

The lateral load-drift hysteretic response of each specimen was plotted using the data recorded at the point of the application of the actuator. The hysteretic responses of both tests, SS1 and SS2, are shown in Fig. 5(a) and 5(b), respectively. As shown in the figures, both specimens display long and narrow hysteresis loops in the drift range from 0.25% to 1.50%, demonstrating a limited ability to dissipate energy. In one cycle of lateral drift, each test specimen behaved similarly to a linear elastic structure with viscous damping. This is similar to that found in the previous tests on the specimens without shear reinforcement as reported in [6]. As the drift level became higher, in general, specimen stiffness degraded more and the hysteresis loops were wider. No significant pinching was observed from the hysteresis loops of either specimen. All specimens experienced punching failure. The punching failure of Specimen SS1 is indicated in Fig. 5(a) by the sudden drop in lateral load capacity after completing two cycles at 3% drift in positive directions. The punching failure was found inside the shear-reinforced zone. On the other hand, SS2 was able to avoid punching shear failure and sustain lateral drifts as high as 4% with no more than a 15% decrease in peak lateral load capacity. The punching failure in SS2 was found outside the shear-reinforced zone. The failure plane suggested that the shear reinforcement in the form of double-head studs in SS2 was effective to prevent the punching shear failure inside the shear-reinforced zone.



(a) Specimen SS1 with closed-hoop stirrups (b) Specimen SS2 with double-head studs

Fig. 5 - Lateral force-drift results

Fig. 6 compares the envelope curves of both PT specimens from this study and the PT specimen without shear reinforcement from [6]. All PT specimens in Fig. 6 were bonded system and designed with the same gravity shear ratio. The specimen S1 was used as the control specimen. As can be seen from the figure, both specimens with shear reinforcement exhibit lateral load-carrying capacity and drift capacity higher than the control specimen without shear reinforcement. The data shows the beneficial effect of both types of shear reinforcement in providing an overall increase.

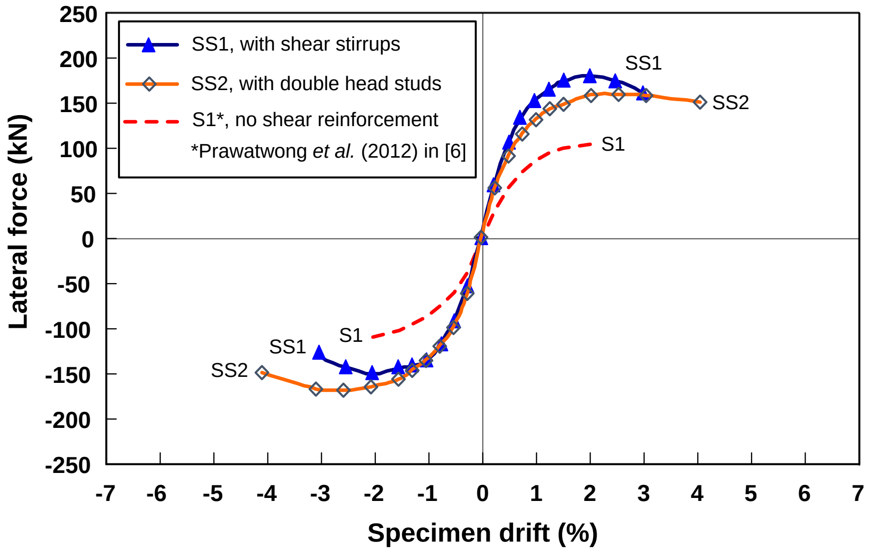


Fig. 6 - Envelope curves of the tested specimens

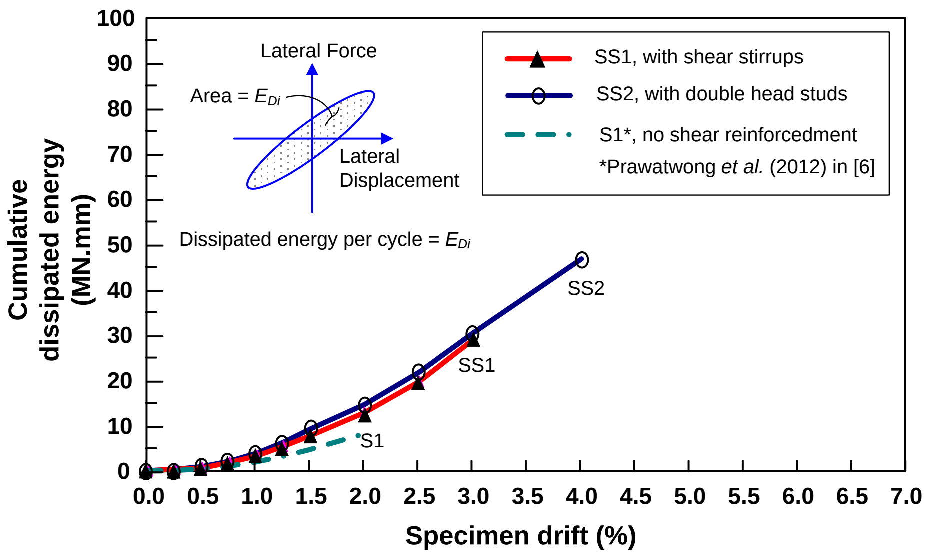
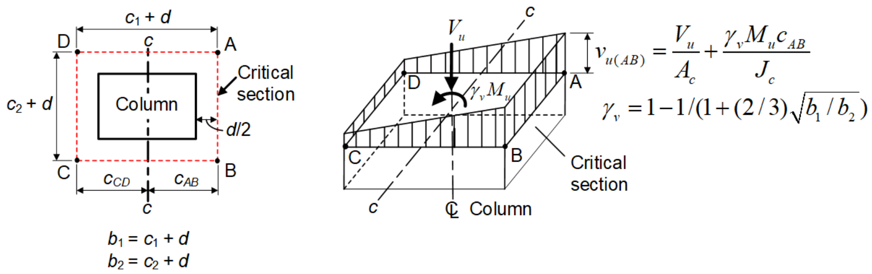


Fig. 7 – Cumulative dissipated energy of the tested specimens

Energy dissipation capacity is an important parameter for evaluating the structure capacity to survive in cyclic loading without collapse. Fig. 7 shows 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 Fig. 7, the specimen with double-head studs exhibited the ability to dissipate energy larger than the specimen without shear reinforcement by almost 575%. In advance of the punching shear occurrence, the specimen with double-head studs was able to dissipate energy up to 46.54 MN.mm, while the one with closed-hoop stirrups was able to dissipate energy up to 28.71 MN.mm.

3.2 Comparison of shear stresses

To compare the increase in punching resistance provided by the different punching shear reinforcement systems, the ACI model for the design of slab-column connections without punching shear reinforcement as shown in Fig. 8 is used to calculate the eccentric shear stress due to a gravity shear V_u and an unbalanced moment M_u along the critical section at $d/2$ from the column face.



(a) critical section (b) stress distribution along critical section.

Fig. 8 – Critical sections at an interior column for linear varying shear stress according to ACI Building Code

The maximum shear stresses at the critical sections are expressed by the well-known equations shown below.

$$v_{u(AB)} = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_{AB}}{J_c} \tag{1}$$

where $A_c = b_0\, d$; $b_0 = 2(b_1 + b_2)$ = perimeter of critical section for shear in slab; d is the effective depth of the slab; c_{AB} is the distance from the centroidal axis of the critical section to line AB (see Fig. 8(a)); J_c is a property of the critical perimeter analogous to the polar moment of inertia; γ_v is the fraction of the unbalanced moment transferred by eccentricity of shear stress and is given in Fig. 8(b).

For each specimen, the unbalanced moment M_u can be accurately determined by multiplying the peak lateral force by the column height (1800 mm) of the specimen. The gravity shear V_u in each specimen is computed from a linear finite element analysis. Based on the peak unbalanced moment (M_u) and the gravity load (V_u) on the test specimens, the maximum shear stresses according to the ACI model for SS1 and SS2 were obtained by Eq. (1) and listed in Columns 7 of Table 2.

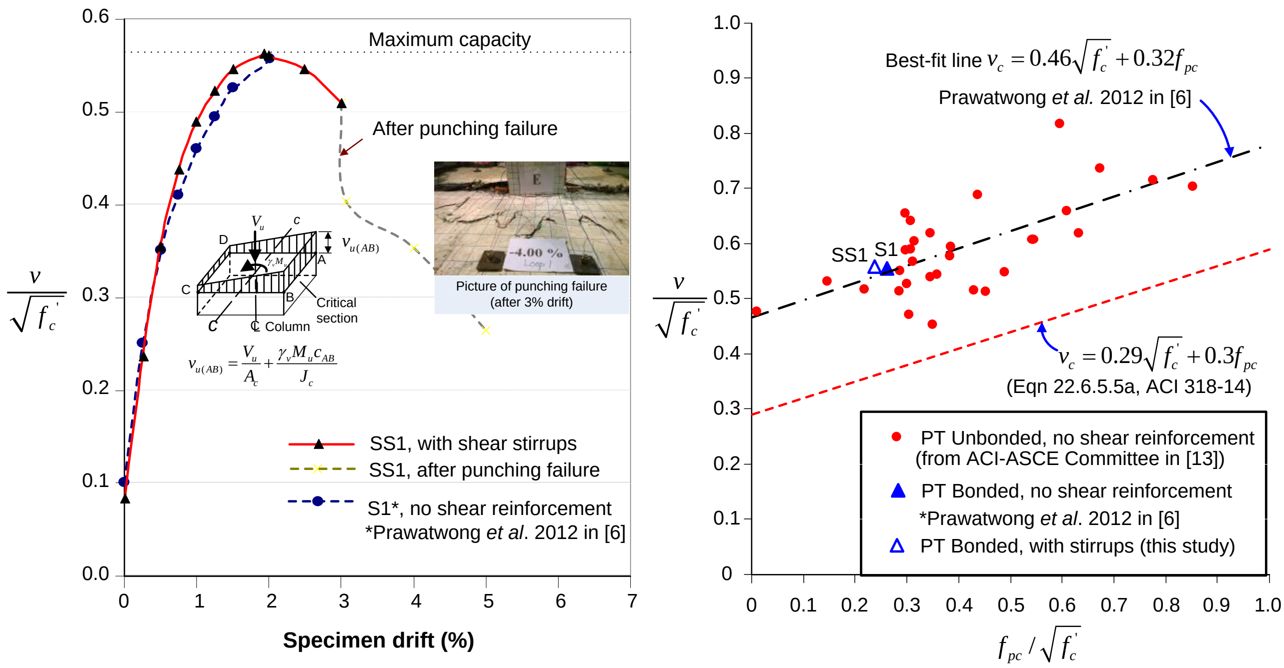
Table 2 – Ultimate Shear Stresses [a]

Specimen	**V_u (kN)**	**M_u (kN.m)**	γ_v	**Shear stress, v_u (MPa)**			$v_u/\sqrt{f'_c}$
				From direct shear	**From moment transfer**	**Total**	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
S1[b]	118	193	0.467	0.65	2.86	3.51	0.55
SS1	141	325	0.463	0.59	3.46	4.05	0.56
SS2	141	288	0.463	0.59	3.07	3.66	0.59

Notes: [a] No load factors were used in calculations. [b] Control specimen with no shear reinforcement from [6].

In Fig. 9(b), the maximum shear stresses v_u of SS1 in Table 2 are plotted and compared with shear stress limits expressed by Eqn (22.6.5.5a) of ACI 318–14 and by previous works from other investigators. Both shear stress v_u and f_{pc} are normalized by the square root of the slab compressive strength in SI units (MPa). The test data from previous works, represented by the red dots, were summarized by ACI-ASCE Committee in [13]. They were mostly obtained from tests conducted for connections transferring shear only, and all tested PT specimens were unbonded flat plate connections that failed in shear. All of them were without shear reinforcement. To determine the true stress limit, an empirical best-fit equation was derived in [6] and depicted in Fig. 9(b). This best-fit equation, therefore, represents the most likely value of shear stress at failure in slab-column connections without punching shear reinforcement.

The comparisons of the ultimate shear stress in Fig. 9(a) and 9(b) pointed out that the ultimate shear stress of SS1 with conventional stirrup reinforcement under lateral cyclic loading was not much increased by the presence of shear reinforcement. However, the results in Fig. 9(a) show that the punching shear failure in SS1 occurred much later after one side of the connection reached the ultimate shear stresses at the drift level of 2%. This implied that the presence of conventional stirrup reinforcement in SS1 was helpful to enhance lateral drift capacity after one side of the connection reached the ultimate shear stress.



Note: 1. Effective depth $d = 0.8h$ was assumed according to ACI 318-02.
2. Combined stresses were calculated at $d/2$ = 60 mm from column face.

(a) Variation of maximum shear stress on critical section of SS1 versus the previous work with no shear reinforcement

(b) Test data of Specimen SS1 versus the other works and ACI 318-14 equation

Fig. 9 – Comparison of shear stresses

3.3 Comparison of drift capacity

Fig. 10 shows a plot of the gravity shear ratio and drift capacity at punching of both specimens from this study, 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 in [14], while those of unbonded PT slab-column interior connections were tested and reported in [15, 3] and summarized in [5]. ACI 318-14 design drift limit for slab-column connections is also plotted in Fig. 10 for reference. For bonded PT slab-column connections, the data from SS1 and SS2 shows the beneficial effect of both types of shear reinforcements in providing an overall increase in the lateral drift capacity for a gravity shear ratio equal to 0.28.

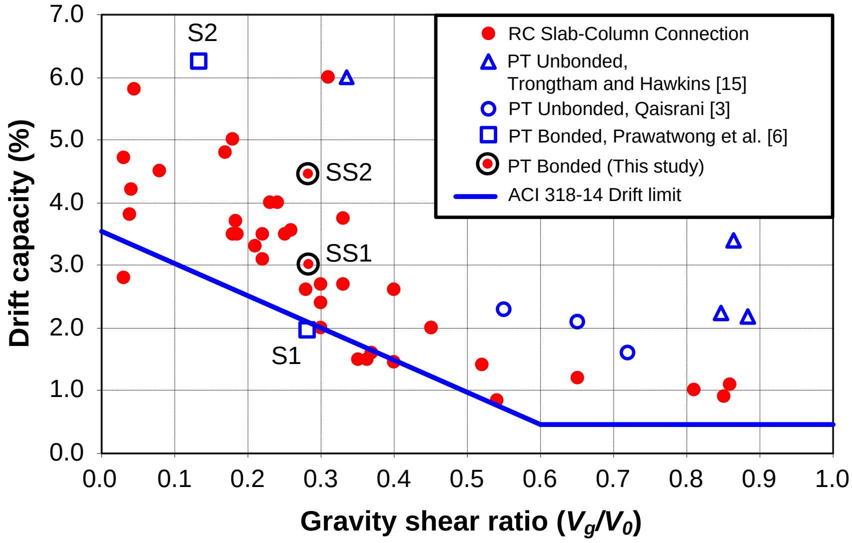


Fig. 10 – Gravity shear ratio versus drift capacity at punching for RC and PT slab-column connections

4. Summary and conclusions

Two three-fifth scale models of bonded PT interior slab-column connections were design and constructed to represent typical details of slab-column connections with shear reinforcements in PT flat plate buildings in Thailand. The models were tested under a conventional reversed cyclic lateral loading until failure to investigate their seismic performance. Based on the results of the experimental investigations conducted on bonded PT interior slab-column connections with different punching shear reinforcement systems and comparing the results with the models without shear reinforcement, the following conclusions can be drawn:

1. During the test, both specimens essentially behaved like a linear elastic system with viscous damping. As the drift level increased, the lateral stiffness of the specimens decreasingly degraded. SS2 with double-head studs exhibited dramatic increases in lateral drift capacity, compared to those of SS1.
2. Punching shear reinforcement in the form of double-head studs effectively and significantly enhances the poor performance of the typical bonded PT interior connections. The specimen SS2 with double head studs showed ductile behavior under reversed cyclic loading. The ductile behavior was clearly demonstrated by its lateral forced-drift relationship.
3. The experimental results from this study pointed out that the conventional shear reinforcement in the form of closed-hoop stirrups may not provide a significant increase in punching shear strength for the thin slab under earthquake type loading. However, the test results suggested that the presence of conventional stirrup reinforcement in SS1 was helpful to enhance energy dissipation capacity and lateral drift capacity.

5. Acknowledgements

The experimental works were conducted with funding provided by Thailand Research Fund (TRF) and National Research Council of Thailand (NRCT). The prestressing strands, ducts and anchors used in the tests were donated by Concrete Products and Aggregate Company (CPAC) Limited. Double head studs were donated by SNP Post-tension co., Ltd. For initial backgrounds data was supported by Postteck Prestressing co., Ltd. Heartfelt gratitude is conveyed to them for their great supports in the research program.

6. Copyrights

7. References

[1] Hawkins NM, Mitchell D. (1979): Progressive collapse of flat-plate structure, *ACI Structural Journal*, **76** (7), 775-809.

[2] Hawkins NM (1981): Lateral Load Resistance of Unbonded Post-Tensioned Flat Plate Construction. *PCI Journal*, **26** (1), 94-116.

[3] Qaisrani AN (1993): Interior Post-Tensioned Flat-Plate Connections Subjected to Vertical and Biaxial Lateral Loading, *PhD thesis, Department of Civil Engineering*, University of California-Berkeley, Berkeley, California, USA.

[4] Gayed RB, Ghali A (2006): Seismic-Resistant Joints of Interior Columns with Prestressed Slabs. *ACI Structural Journal*, **103** (5), 710-719.

[5] Kang THK, Wallace JW (2006): Punching of Reinforced and Post-Tensioned Concrete Slab-Column Connections, *ACI Structural Journal*, **103** (4), 531-540.

[6] Prawatwong U, Warnitchai, P, Tandian, CH (2012): Seismic Performance of Bonded Post-tensioned Slab-column Connections with and without Drop Panel, *Advances in Structural Engineering*, **15** (10), 1653-1672.

[7] Yan Zhou, Marry Beth D. Hueste. (2016): Review of test data for interior slab-column connections with moment transfer, *ACI-fib International symposium punching shear of structural concrete slab*, 141-166.

[8] ACI Committee 421.1R-08. (2008): Guide to shear reinforcement for slab, *American concrete institute*, Farmington Hills, USA.

[9] ASTM A1044/A1044M-16a. (2016): Standard specification for steel stud assemblies for shear reinforcement of concrete.

[10] Pan AD, Moehle JP (1988): Reinforced Concrete Flat Plates under Lateral Loading: An Experimental Study Including Biaxial Effects, *Report UCB/EERC-88/16*, Earthquake Engineering Research Center, University of California, Berkeley, USA.

[11] ACI Committee 318 (2014). Building code requirements requirement for structural concrete and commentary, *American concrete institute*, Farmington Hills, USA.

[12] Luckkikanun A, Prawatwong U, Warnitchai P (2019): Cyclic testing of bonded post-tensioned interior slab-column connections with shear stud. *16th East Asia-Pacific Conference on Structural Engineering & Construction (EASEC16)*, Brisbane, Australia.

[13] ACI-ASCE Committee 423 (1974): Tentative recommendations for prestressed concrete flat plates. *American concrete institute*, **71** (2), 61-71.

[14] Pan AD, Moehle JP, (1989): Lateral Displacement Ductility of Reinforced Concrete Flat Plates, *ACI Structural Journal*, **86** (3), 250-258.

[15] Trongtham N, Hawkins NM (1977): Moment Transfer to Columns in Unbonded Post-Tensioned Prestressed Concrete Slabs, *Report SM77-3*, *Department of Civil Engineering*, University of Washington-Seattle, Seattle, Washington, USA.

