SEISMIC PERFORMANCE OF POST-TENSIONED INTERIOR FLAT SLAB-COLUMN CONNECTIONS

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ABSTRACT

This paper presents the results of reversed cyclic loading test of a 3/5 scale slab-column connection model, which was carefully design and constructed to represent a typical connection between interior column and post-tensioned flat slab with bonded tendons in Thailand. A conventional displacement-controlled cyclic loading test with monotonically increasing drift levels until failure was adopted to investigate the seismic performance of the connection. The lateral force-deformation relation indicated that the connection model essentially behaved like a linear elastic system with low energy dissipation. As the drift level increased, cracks on the slab surface grew in size and number and concentrated around the column, and the lateral stiffness of the model degraded significantly. Shortly after attaining its maximum lateral strength at 2% drift, the specimen abruptly failed by punching shear. The drift at which the non-ductile failure occurred is considered to be rather low, and hence design improvement for slab-column connections is deemed desirable. The test results on cyclic properties of the response, including stiffness degradation, hysteretic shape, and failure mode, will be useful for the evaluation of seismic performance of the entire slab-column frame buildings in the future.

1. BACKGROUND

Over the past three decades, rapid urbanization and massive scale of building construction have taken place in Bangkok and several major cities in Thailand. As the country has long been considered as being free from seismic risk, most existing buildings have been designed and constructed without any consideration on seismic loading. Recently, however, there has been a significant improvement in the understanding of seismic risk. New probabilistic seismic hazard studies indicate that northern and western Thailand can be regarded as regions of moderate seismic hazard, and that Bangkok, though located at a remote distance from seismic sources, is still at risk from long-period, damaging ground motions induced by distant large earthquakes (Warnitchai 2004). The risk in Bangkok is primarily caused by the ability of thick soft surficial deposits in the city area to amplify earthquake ground motions about 3 to 4 times.

To mitigate the risk, seismic design requirements in the form of mandatory ministerial regulations were introduced in 1997. The regulations stipulate that public buildings, essential facilities, hazardous facilities, and all other buildings with height above 15 meters in 10 provinces in northern and western Thailand must be designed for a moderate level of earthquake ground shaking. A revision of the regulations is currently being made to include the design requirements against the effects of distant large earthquakes for buildings in Bangkok and 4 neighboring provinces.

Despite the introduction of statutory seismic design requirements, their actual implementation in design practices seems to have many difficulties and limitations. Most design engineers are not familiar with seismic design concepts and procedures, and they normally do not understand the need for seismic detailing. Many engineers believe that buildings typically designed for gravity loads and wind load (but no seismic detailing) do have sufficient inherent capacity to withstand the expected moderate earthquake ground shaking.

Under these circumstances, a research program on the seismic performance of several typical buildings in Thailand is currently being conducted by the authors. The main objectives are to: (1) determine the inherent seismic capacity of typical buildings of various forms; (2) identify their typical weak spots, detailing deficiencies, and poorly performed structural configurations; and (3) find out economic and practical ways to improve the design of new buildings and to retrofit existing buildings. One key element in this research program is a study on the performance of some critical building components under reversed cyclic loading. These components include, for example, RC columns with short lap splices, beam-column joints with no joint reinforcement, and slab-column connections, etc.

In this paper, a study on the seismic performance of post-tensioned interior flat slab-column connections is presented. A quasi-static, reversed cyclic loading test of a 3/5 scale connection model was carried out, and its behavior and failure mode were examined in detail. The results from this study will be critical ingredients for the evaluation of seismic performance of slab-column frame buildings in Thailand.

2. SLAB-COLUMN FRAME BUILDINGS IN THAILAND

Post-tensioned flat slab construction is popular in Thailand for medium to high rise buildings such as office buildings, hospitals, residential buildings and parking buildings. A slab-column frame is normally designed to carry only gravity loads, while the lateral wind load is assumed to be taken care of by concrete shear walls. The slab-column frame is neither designed for lateral seismic load nor checked for lateral deformation compatibility with shear walls to ensure that it can undergo the maximum lateral drift (due to seismic load) without losing gravity load carrying capacity.

It is widely known that the slab-column connection is a critical component in the slab-column frame system. This is the region of slab immediately adjacent to the column that has to transmit large torsion, shear and bending moments between slab and column and is therefore susceptible to punching shear failure. In Thailand, slab-column connections are typically not designed and detailed for seismic effects. No shear reinforcement (such as stirrups or stud-rails) is provided at slab-column connections. Although slab bottom reinforcement bars are provided in an orthogonal mesh to satisfy a minimum requirement for temperature and shrinkage effects, there may be no continuous bottom bar passing through the column to protect against progressive collapse after punching shear failure. Furthermore, due to the congestion of reinforcement bars in the column section, prestressing tendons are normally arranged such that none of them passes through the column.

Despite the fact that a reasonably large number of research studies on the behavior of slab-column connections under seismic loading have been carried out in the past, most of them were made for reinforced concrete slab-column connections. Only a few focused on post-tensioned flat slab-column connections (e.g. Hawkins 1981, Burns et al. 1985, Martinez-Cruzado et al. 1994, Kang et al. 2004), and none of them were made for 'bonded' post-tensioning tendons system with non-seismic reinforcement detailing, which is the prevailing type of flat slab construction in Thailand.

3. KEY STRUCTURAL INDICES

As the objective is to study the seismic performance of post-tensioned slab-column connections that are typical in Thailand, an effort was made to acquire architectural and structural drawings of five representative buildings with post-tensioned floors in Bangkok. The number of stories of these buildings varies from 15 to 30. Some important structural parameters associated with seismic behavior are computed from the drawings; they are herein called 'structural indices'. These indices are: gravity shear ratio (V_g / V_0), critical section perimeter-to-depth ratio (b_0 / d), side ratio (b_1 / b_2), gravity shear-to-moment ratio ($e_r V_g / M_g$), prestressing ratio ($f_{pc}/\sqrt{f_c'}$), negative moment reinforcement ratio (ρ_s), and gravity moment ratio (M_g / M_n).

In the above, V_g is the gravity shear acting on the slab critical section, V_0 is the direct punching shear strength as defined by ACI 318-95, b_0 is perimeter of the critical section, d is the effective depth, b_1 is the width of the critical section measured along the direction of loading, b_2 is the other dimension of the section orthogonal to b_1, M_g is the negative moment at the slab-column connection caused by gravity load, e_r is the ratio of the shear stress caused by a unit direct shear to the maximum shear stress caused by a unit unbalanced moment on the critical section, f_{pc} is the compressive stress in concrete slab at the centroid of cross section due to prestressing force, ρ_s is the ratio of total area of top reinforcement bars to $(c_2 + 3d).d$, c_2 is the column width measured orthogonal to the direction of loading, M_n is the nominal moment capacity. Note that the gravity load here is the dead load (without load factor) plus 'likely live load' of which the value is given by ATC-40 (ATC 1996) based on the building occupancy type.

The indices' values of interior slab-column connections in five representative buildings are given in Table 1. The values do not vary much from case to case, indicating the structural similarity between all these cases. Among these indices, the gravity shear ratio (V_g / V_0) appears to be the most important one, as many test results in the past indicate that the lateral drift level at which a connection punching shear occurs is strongly influenced by this index (Hueste et al., 1999). For gravity shear ratios higher than 0.4—which is the maximum limit recommended by ACI 318-95, the drift capacity could be unacceptably low, say below 1.5 % in some cases. Table 1 shows that the gravity shear ratio of every representative building falls within the limit, with a moderately high average value of 0.289.

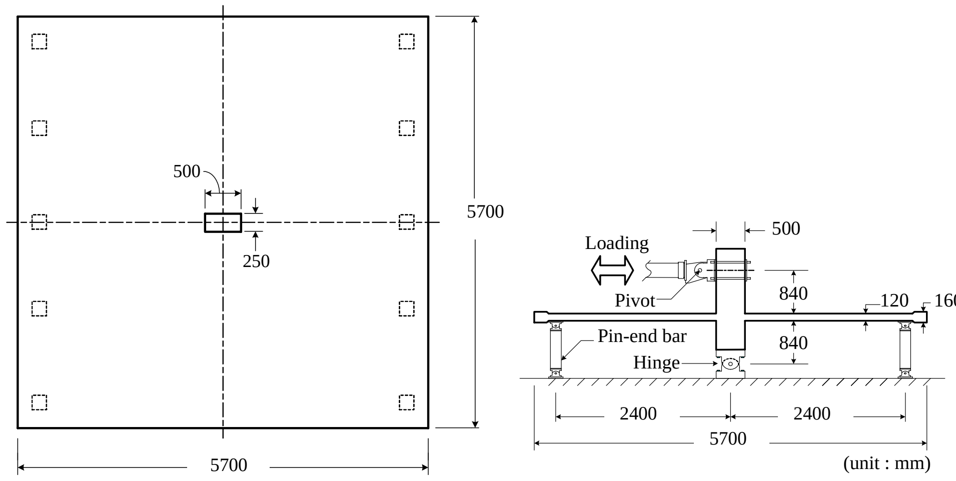
Table 1: Structural indices of slab-column connections in five representative buildings and those of test specimen

Building	**Span** (cm)	**Column Size** (cm x cm)	$\frac{V_g}{V_0}$	$\frac{b_0}{d}$	$\frac{b_1}{b_2}$	$\frac{e_r V_g}{M_g}$	$\frac{f_{pc}^{*}}{\sqrt{f_c'}}$	ρ_s	$\frac{M_g}{M_n}$
Office 1	800	40x100	0.267	21.5	2.07	1.72	0.84	0.014	0.13
Office 2	700	40x80	0.296	19.0	1.71	1.67	1.01	0.011	0.11
Office 3	800	40x80	0.287	19.0	1.71	1.21	0.84	0.010	0.16
University	800	50x80	0.231	18.8	1.44	1.40	1.04	0.008	0.11
Hospital	840	50x70	0.366	18.3	1.30	1.52	0.87	0.009	0.15
Average	800	40x80	0.289	19.3	1.65	1.50	0.92	0.010	0.13
Specimen	480	25x50	0.280	18.3	1.70	3.25	0.93	0.011	0.07

* *unit = kg/cm²*

4. TEST SPECIMEN AND EXPERIMENTAL SETUP

The test specimen is a 3/5 scale slab-column interior connection model as shown in Figure 1. As the inflection points in the slab-column frame system under seismic loading are assumed to occur at slab mid-span and column mid-height, the model slab extends to mid-span on two sides of the connection and the column extends above and below the slab to story mid-height. The slab was supported along each transverse edge by 5 pin-ended bars to simulate a moment-free boundary condition. Similarly, to produce moment-free condition at the column ends, the column bottom end was set on a hinged support, and the top end was connected to a hydraulic actuator through a pivoted connection. The test specimen was designed to have the values of structural indices close to the average values of representative buildings as shown in Table 1.



(a) Plan *(b) Elevation*

Figure 1: Interior slab-column connection specimen and its dimensions

The prestressing strands are grade 270, seven-wire, stress relieved type with a nominal diameter of 12.7 mm. Eight strands were banded in the direction of loading with a spacing of 350 mm. Other eight strands were placed uniformly in the direction perpendicular to the loading with a spacing of 700 mm as shown in Figure 2. Each strand was inserted into a galvanized duct to prevent bonding with concrete before prestressing. Three days after casting slab, all these strands were tensioned one by one to about 80 % of their ultimate strength (0.80 f_{pu}). Shortly afterward, all galvanized ducts were filled in by non-shrink cement grout.

Top slab reinforcement bars were placed symmetric about both centerline axes as shown in Figure 3. The top bars were concentrated only at the slab-column connection region and have a spacing of 80 mm. These bars were cut off at a distance of 1.0 m from the center of the column. Figure 3 also shows the layout of bottom slab reinforcement which is symmetric about both center line axes. The bottom bars were spaced at 550 mm intervals throughout the slab. Although the specified steel grade of all slab reinforcement bars was SD-30, their tested yield and tensile strengths were about 440 and 580 MPa, respectively. The average compressive strength of concrete cylinders for slab at 4, 14, and 28 days were 20, 39, and 40 MPa, respectively.

Before applying a lateral reversed cyclic load, a large number of sand bags were piled up on and hanged underneath the slab as shown in Figure 4 in order to correctly simulate the gravity load effects. The amount and distribution of sand bags were determined by a finite element analysis such that the computed gravity shear ratio was close to the average value of representative buildings. The lateral load was applied to the top column by an MTS servo controlled hydraulic actuator mounted horizontally to a rigid reaction wall (Figure 4).

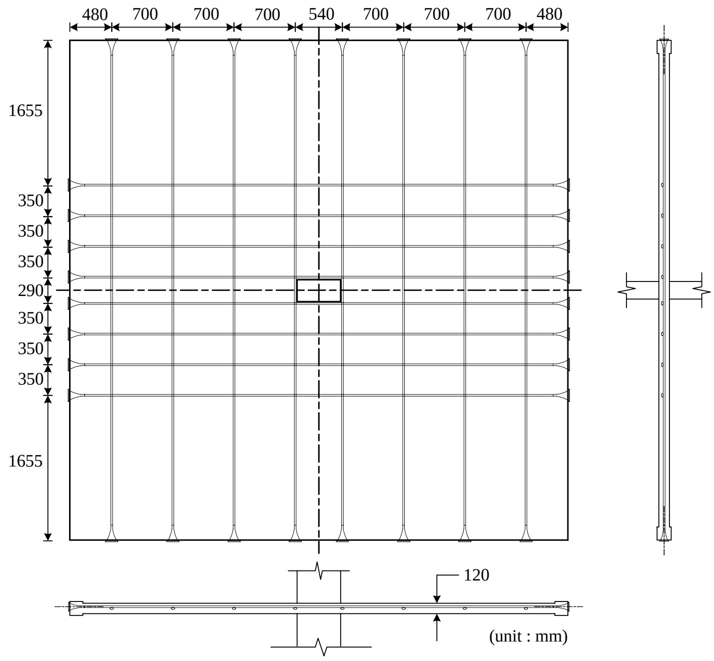


Figure 2: Layout of prestressing strands

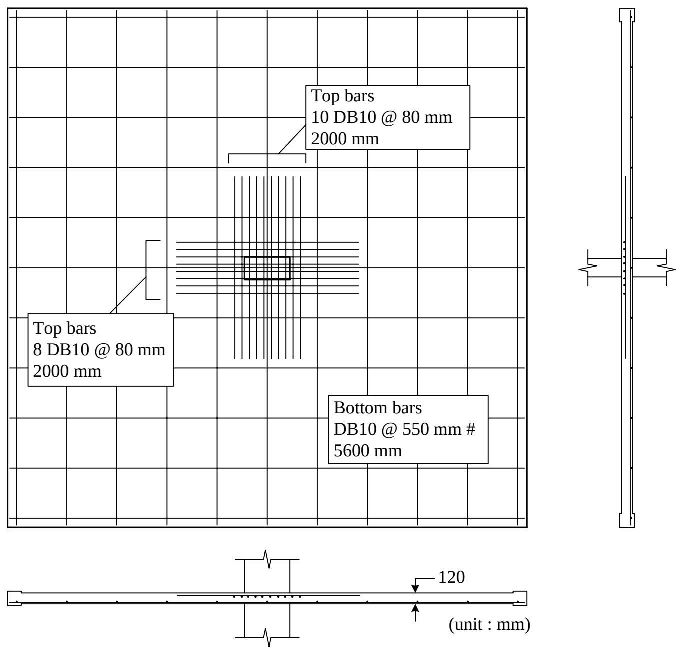


Figure 3: Layout of top and bottom steel deformed bars

Note that this test setup was found to be rather weak in torsion, so a torsional restraining system was attached to the test specimen. A typical displacement-controlled cyclic loading test was then carried out with monotonically increasing drift levels of ±0.25%, ±0.5%, ±0.75%, ±1.00%, ±1.25%, ±1.50%, ±2.00%, …. For each drift level, two complete cyclic displacement loops were made.

Figure 4: Setup for reversed cyclic loading test

The data measured and recorded in the experiment include: (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, (4) strain in top and bottom bars of slab at various locations, (5) strain distribution along some prestressing strands, and (6) strain of longitudinal bars of column. Photos were also taken at peak positive and negative drifts in every cycle of loading to record the development of visible cracks on top slab surface in the connection region. Full details of the test specimen instrumentation can be found in (Pongpornsup 2003).

4. EXPERIMENTAL RESULTS

Due to space limitation, only some results are presented here in this section. First, the development of cracks on the top surface of the slab around the column is shown in Figure 5. The first observable cracks were longitudinal cracks running in the direction of loading and passing through the column sides. They initiated at the lateral drift of 0.25%. The development of diagonal cracks radiated from column corners followed afterward and became more obvious at 0.5% drift. These diagonal cracks might be caused torsion in the slab, which was resulted from the difference in flexural deformation of slab strips near and far from the column faces. The flexural deformation of the slab strip adjacent to the column face was found to be the highest. Transverse cracks were clearly developed at about 1.0% drift. As the drift level increased, these longitudinal, diagonal, and transverse cracks widened and lengthened and grew in number around the connection.

While the slab was pushing toward the positive direction after completing two cycles at 2.0% drift, a punching shear failure suddenly occurred on one side of the connection at about 1.70% drift. After that, the slab was then pulling back toward the negative direction, and another punching shear failure suddenly developed on the other side of the connection at about -0.8% drift, thus forming a complete loop of punching shear failure as shown in Figure 5. Note that the punching shear failure did not occur at the critical section.

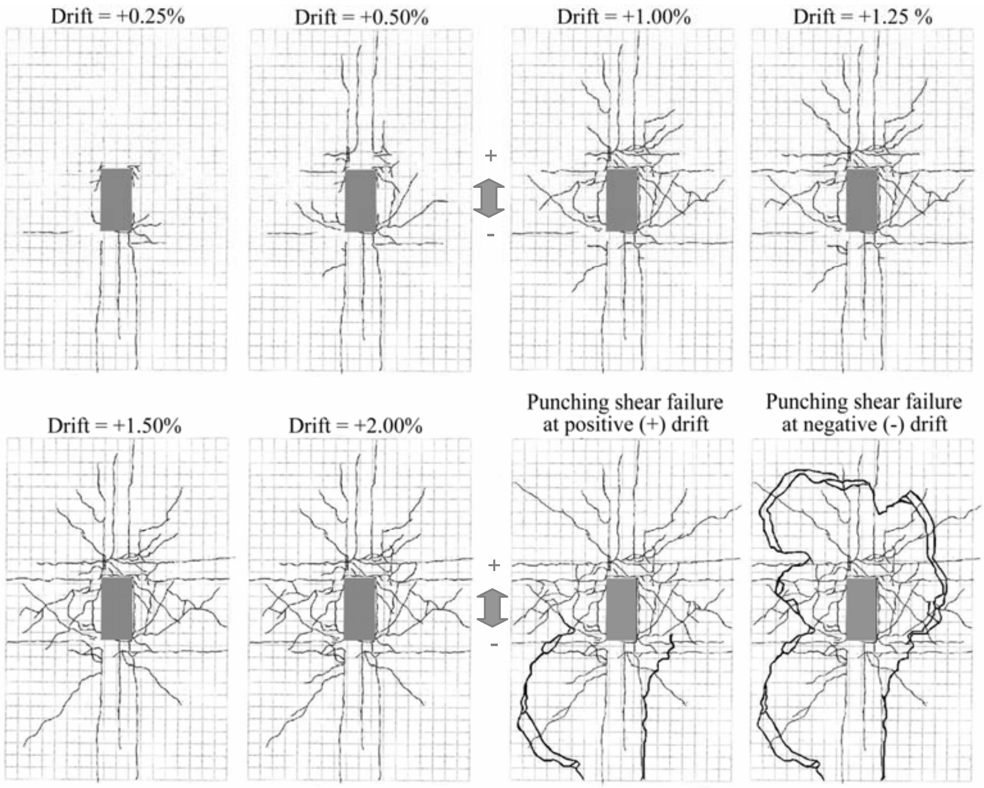


Figure 5: Development of cracks on the top surface of slab

The relation between lateral load and lateral drift is shown in Figure 6. The hysteretic loop in every loading cycle before punching shear failure was long and narrow, indicating a limited ability to dissipate energy. Neither pinching behavior nor plastic residual deformations were observed. As the drift level increased, the peak lateral load also increased, but the average stiffness (secant stiffness) reduced. The test specimen essentially behaved like a linear elastic system with significant stiffness degradation. The stiffness degradation is believed to be caused by the extensive and progressive cracking of slab in the connection region. Strains in slab bottom bars, prestressing strands, and column longitudinal bars fluctuated within the limit of linear elastic, while strains in slab top bars exceeded the yield limit at about 1.5% drift. The maximum lateral load of 105 kN was attained at 2.0% drift. After the punching shear failure occurred at around 2.0% drift, the test specimen completely lost its lateral strength and stiffness.

Compared with other test results on slab-column connections done elsewhere (e.g. Hueste 1999), the drift of about 2.0% at which the punching shear failure occurred appears to be rather low. Improvement in the design of post-tensioned slab-column connections is deemed desirable.

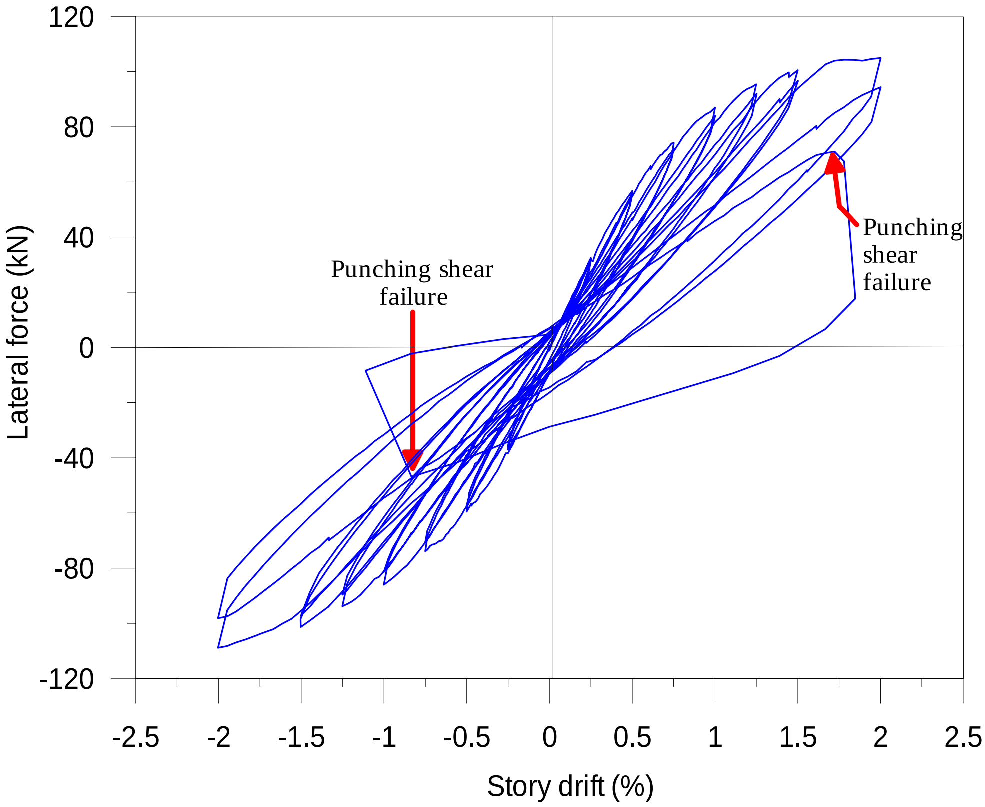


Figure 7: Relation between lateral load and lateral drift

7. CONCLUSIONS

A 3/5 scale model was designed and constructed to represent a typical connection between interior column and post-tensioned flat slab in medium to high rise buildings in Thailand. The model was tested under a conventional reversed cyclic loading with monotonically increasing drift levels until failure to investigate its seismic performance. During the test, the specimen essentially behaved like a linear elastic system with low energy dissipation, as indicated by its long and narrow hysteretic loops. As the drift level increased, cracks on the slab surface around the column grew in size and number, and the lateral stiffness of the specimen degraded significantly. Shortly after attaining its maximum lateral strength at 2% drift, the specimen abruptly failed by punching shear. The drift at which the non-ductile failure occurred is considered to be rather low, and hence design improvement for slab-column connections is deemed desirable. The test results on cyclic properties of the response, including stiffness degradation, hysteretic shape, and failure mode, will undoubtedly be useful for the evaluation of seismic performance of the entire slab-column frame buildings in the future.

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