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# Seismic capacity evaluation of post-tensioned concrete slab-column frame buildings by pushover analysis

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## Abstract

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**Seismic capacity evaluation of post-tensioned concrete slab-column frame buildings by pushover analysis**
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**Seismic capacity evaluation of post-tensioned concrete slab-column frame buildings designed only for gravity loads and wind load is presented. The series of nonlinear pushover analyses are carried out by using the computer program SAP2000. An equivalent frame model with explicit transverse torsional members is introduced for modeling slab-column connections. The analyses are carried out by following guidelines in ATC-40 and FEMA-273/274, where several important factors such as P-Delta effects, strength and stiffness contributions from masonry infill walls, and foundation flexibility are well taken into account. The pushover analysis results, presented in the form of capacity curves, are compared with the seismic demand from the expected earthquake ground motion for Bangkok and then the seismic performance can be evaluated. Numerical examples are performed on the 9- and 30-storey post-tension flat-plate buildings in Bangkok. The results show that in general post-tensioned concrete slab-column frame buildings without shear wall possess relatively low lateral stiffness, low lateral strength capacity, and poor inelastic response characteristics. The evaluation also shows that the slab-column frame combined with the shear wall system and drop panel can increase the strength and stiffness significantly.**

**Key words :** seismic evaluation, pushover analysis, concrete buildings, slab-column frames

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## บทคัดย่อ

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**การประเมินความสามารถต้านทานแรงแผ่นดินไหวของอาคารคอนกรีตเสริมเหล็ก ชนิดแผ่นพื้นไร้คาน โดยวิธี Pushover Analysis**

**ว. สงขลานครินทร์ วทท. 2549 28(5) :**

งานวิจัยนี้ได้เสนอวิธีการประเมินความสามารถต้านทานแรงแผ่นดินไหวของอาคารคอนกรีตเสริมเหล็กชนิดแผ่นพื้นไร้คานที่ออกแบบไว้สำหรับน้ำหนักในแนวดิ่งและแรงลม โดยใช้การวิเคราะห์วิธี Nonlinear Static Pushover โดยโปรแกรม SAP2000 แบบจำลองโครงข้อแข็งชนิดแผ่นพื้นไร้คานเทียบเท่าได้ใช้วิธี Explicit transverse torsional member ในการจำลองจุดต่อของพื้น-เสา การวิเคราะห์ได้ใช้ตามข้อแนะนำของ ATC-40 และ FEMA-273/274 โดยพิจารณาผลของ P-Delta Rigid Zone กำแพงอิฐก่อ และฐานราก ผลการวิเคราะห์ Nonlinear Static Pushover แสดงในรูปแบบของความสัมพันธ์ระหว่างแรงเฉือนที่ฐานอาคารกับการเคลื่อนตัวด้านข้างของยอดอาคาร หรือเรียกว่า Capacity Curve ส่วนการประเมินได้เสนอการประเมินอาคารโดยวิธี Capacity Spectrum โดยใช้การคำนวณ Capacity Curve ร่วมกับการคำนวณ Demand Curve ที่แสดงระดับกำลังต้านทานและความเหนียวที่อาคารจำเป็นต้องมีภายใต้แผ่นดินไหวที่จำลองขึ้นสำหรับกรุงเทพมหานคร ตัวอย่างการคำนวณและการประเมินได้ใช้อาคารคอนกรีตเสริมเหล็กชนิดแผ่นพื้นไร้คานสูง 9 ชั้น และ 30 ชั้น ในกรุงเทพฯ ผลการวิจัยพบว่าอาคารคอนกรีตเสริมเหล็กชนิดแผ่นพื้นไร้คานที่ไม่มีผนังแรงเฉือน โดยทั่วไปมีความแข็งแรงและกำลังต่ำ และพฤติกรรมในช่วงอินอิลาสติกไม่ดี การประเมินยังพบว่าอาคารคอนกรีตเสริมเหล็กชนิดแผ่นพื้นไร้คานที่มีผนังแรงเฉือน และเพิ่มความแข็งแรงบริเวณหัวเสา สามารถเพิ่มความแข็งแรงและกำลังต้านทานแรงด้านข้างได้เป็นอย่างมาก

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The Sumatra earthquake with a Richter magnitude of 9 and tsunami on December 26, 2004 caused large destruction and killed more than 300,000 people in 6 countries including Thailand. The epicenter was about 1,200 km from Bangkok. The series of moderate aftershocks with a Richter magnitude of about 6 was 800 to 1,200 km from Bangkok. A moderate earthquake at a distance of 200 to 400 km from Bangkok may occur sometime in the future. Therefore, Bangkok, the capital of Thailand, is at moderate risk of distant earthquake due to the ability of soft soil to amplify ground motion about 3-4 times. In addition, before the enforcement of seismic load for building in northern of Thailand in 1997 and even now, many existing post-tensioned concrete slab-column buildings in Bangkok may have been designed without consideration for seismic loading. Therefore, the evaluation of seismic capacity of existing buildings in Bangkok is needed. The static push-over procedure has been presented and developed over the past twenty year (Saiidi and Sozen, 1981; Fajfar and Gaspersic, 1996; Bracci *et al*., 1997; Krawinkler and Seneviratna, 1998; Kiattivisanchai, 2001; Imarb, 2002). The method is also described and recommended as a tool for design and assessment purposes by the National Earthquake Hazard Reduction Program 'NEHRP' guidelines for seismic rehabilitation of existing buildings and by the Applied Technology Council (1996) guidelines for seismic evaluation and retrofit of concrete buildings. Moreover, the technique is accepted by the Structural Engineers Association of California 'SEAOC Vision 2000' (1995) among other analysis procedures with various levels of complexity. This analysis procedure is selected for its applicability to performance-based seismic design approaches and can be used at different design levels to verify the performance targets. Finally, it is clear from recent discussions in code-drafting committees in Europe that this approach is likely to be recommended in future codes.

Although the static pushover procedure has been presented for seismic capacity evaluation of reinforce-concrete beam-column frames by many researches (Saiidi and Sozen, 1981; Fajfar and Gaspersic, 1996; Bracci *et al.*,1997; Krawinkler and Seneviratna, 1998; Kiattivissanchai, 2001; Imarb, 2002) very few researches have studied seismic capacity evaluation of post-tensioned concrete slab-column frames. This paper presents a study of inherent seismic capacity of post-tensioned concrete slab-column frame buildings designed only for gravity loads and wind load. The series of nonlinear pushover analyses were carried out by using the computer program SAP2000.

## Seismic capacity evaluation method

### Pushover analysis

The nonlinear static pushover analysis is a sample option for estimating the strength capacity of building in the post-elastic range. The technique can also be used to highlight potential weak areas in the structure. This procedure involves applying a predefined lateral load pattern that is distributed along the building height. The lateral forces are then monotonically increased in constant proportion with a displacement control at the top of the building until a certain level of deformation is reached. The target top displacement may be the deformation expected in the design earthquake in case of designing a new structure or the drift corresponding to structural collapse for assessment purposes. The method allows tracing the sequence of yielding and failure on the member and the structure level as well as the progress of the overall capacity curve of the structure.

In this study, nonlinear static pushover analysis is performed by using SAP2000 software (SAP 2000). The SAP2000 software is a three-dimensional static and dynamic finite element analysis and design of structure program which allows for strength and stiffness degradation in the components by providing the force-deformation criteria for hinges used in pushover analysis. More than one type of hinge can exist at the same location to simulate many possible failure modes in the components. The values used to define the force-deformation curve for pushover hinge are very dependent on the type of component, failure mechanism, ratio of reinforcement and many parameters which are described in the ATC-40 (1996) and FEMA-273 ( 1997).

### Acceleration-displacement response spectra

The application of the capacity spectrum technique requires that both the demand response spectra and structural capacity (or pushover) curve be plotted in the spectral acceleration, $S_a$ and spectral displacement, $S_d$ domain, or the so-called Acceleration-Displacement Response Spectra (ADRS).

To construct the capacity spectrum, capacity curve of the multi-storey building is converted into the capacity curve of the equivalent single degree of freedom (SDOF) systems based on the capacity curve, which in terms of base shear and lateral roof displacement, is obtained from push-over analysis (Figure 1a). Any point ($V$ and $\Delta_{roof}$) on the capacity curve is converted to the corres-ponding point $S_a$ and $S_d$ on the capacity spectrum by using the equations (1) and (2)

$$S_a = \frac{V/W}{\alpha_1} \tag{1}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{1,roof}} \tag{2}$$

where $S_a$ = structural acceleration; $S_d$ = structural displacement; $PF_1$ = modal participation factor for the first natural mode; $\alpha_1$ = modal mass co-efficient for the first natural mode; $V$ = base shear; $W$ = building dead weight plus likely live loads; $\Delta_{roof}$ = lateral roof displacement; $\phi_{1,roof}$ = amplitude of the first natural mode at roof.

In this study, the constant ductility yield strength demand spectra are constructed based on a lump mass SDOF system. To compare with the capacity spectrum, every point on a demand

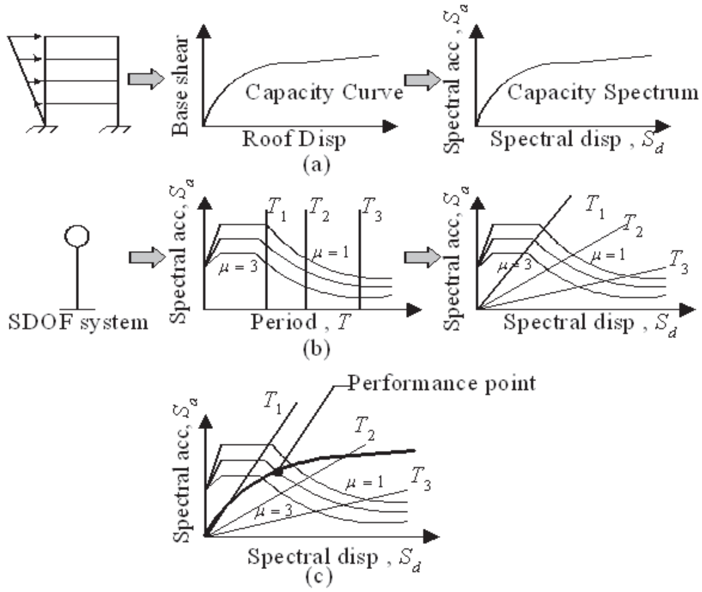



**Figure 1. (a) capacity spectrum, (b) demand spectrum, and (c) capacity spectrum superimposed over demand spectrum in ADRS format**

spectrum will be transformed from the standard $S_a$ versus $T$ format to ADRS format which can be done by using the equation (3).

$$S_d = \left[ \left( \frac{T}{2\pi} \right)^2 S_a g \right] \mu \quad (3)$$

In the ADRS format, line radiating from the origin have constant period. For any point on the demand spectrum in ADRS format, the period, $T$, can be computed by using the equation (4).

$$T = 2\pi \sqrt{\frac{S_d}{S_a g \mu}} \quad (4)$$

The demand spectra in both standard format and ADRS format are presented in (Figure 1b). The intersection between capacity and demand spectrum having the same ductility factor, $\mu$, is the performance point (Figure 1c) that represents the maximum structural force and displacement expected for the demand earthquake ground motion.

**Demand spectrum**

To compare with the capacity spectrum, the demand spectrum obtained from the constant-ductility inelastic response spectrum is plotted in ADRS format. The constant-ductility inelastic response spectrum is a plot of yield strength of lump mass SDOF system as a function of natural period. The yield strength demand is the strength required to limit the displacement to a specified displacement ductility ratio. The displacement ductility ratio is defined as maximum absolute value of the displacement normalized by the yield displacement of the system. The displacement ductility gives a simple quantitative indication of the severity of the peak displacement relative to the displacement necessary to initiate yielding.

Due to the lack of actual recorded ground motions in Thailand, the ground motions records have to be simulated in this study to construct the constant-ductility inelastic response spectra for Bangkok. Some strong ground motion records are selected from actual seismograms from far-field sites. These ground motions are used as input rock outcrop earthquake motions and they are scaled to the required intensity. In this study, the peak rock outcrop acceleration for Bangkok for 50% (100-year return period), 10% (500-year return period), 5% (1,000-year return period), and 2% (2,500-year return period) probability of exceedance in a 50-year exposure period are 0.019g, 0.043g, 0.056g, and 0.075g, respectively, as suggested by Warnichai *et al.* (2000). In addition, when the amplification effect on Bangkok's soft soils is considered, the peak ground accelerations are 0.072g, 0.14g, 0.18g, and 0.22g, respectively, as also suggested by Warnichai *et al.* (2000). It should be noted that the soft soil of Bangkok can amplify ground motion about 3-4 times. In this study, two types of ground motions, namely, scaled ground motion of El Centro, California, during the Imperial Valley earthquake of May 18, 1940, and simulated ground motions for Bangkok are applied. Figure 2 shows ground motion for a 500-year return period for scaled ground motion of El Centro and simulated ground motion for Bangkok.

## Building model

### Model of slab-column frame

An improvement in analyzing reinforced-concrete slab-column frame model, termed the explicit transverse torsional member method was proposed by Cano and Klingner (1988) and is applied in this study. Model of slab-column frame is shown in Figure 3. Conventional columns are connected indirectly by two conventional slab-beam elements, each with half the stiffness of the actual slab-beam. The indirect connection, using explicit transverse torsional members permits the modeling of moment leakage as well as slab torsional flexibility. While the resulting frame is non-planar, this is not a serious complication. Because the transverse torsional members are presented only for the analytical model, their lengths can be taken arbitrarily, as long as the torsional stiffness is consistent.

In explicit transverse torsional member model, gross member properties are used for slab-beams and column. Area, moment of inertia, and shear area are calculated conventionally. For computer input, the torsional stiffness $K_t$ of the transverse torsional members is calculated by using the equation (5)

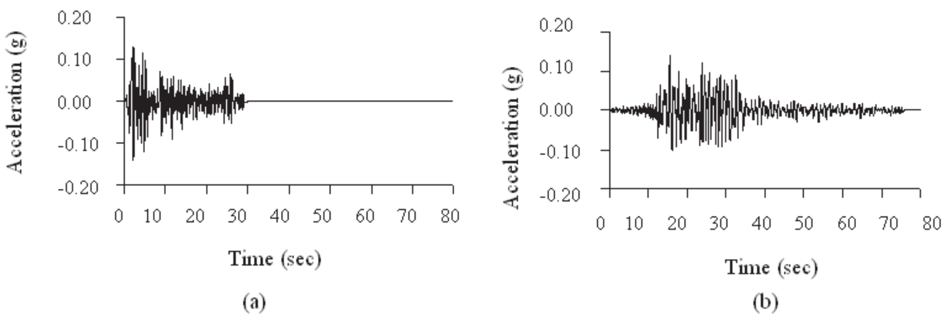



**Figure 2. Ground motion in a 500-year return period (a) scaled ground motion of EI Centro, (b) simulated ground motion of Bangkok**

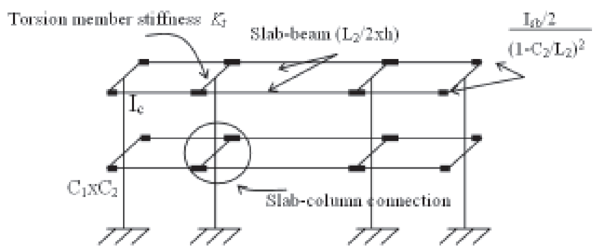



**Figure 3. Model of slab-column frame building**

$$K_t = \sum \frac{9E_{cs}C_t}{l_2\left(1-\frac{c_2}{l_2}\right)} \quad (5)$$

where $E_{cs}$ is the modulus of elasticity of slab concrete, $c_2$ is the dimension of the column in the transverse span of the framing direction, $l_2$ is the transverse span of framing, and $C_t$ is the torsional constant.

Using an arbitrary length $L$ for the torsional members, the torsional stiffness $J$ is then calculated by using the equation (6)

$$J = K_t L / G \quad (6)$$

where $G$ is the shear modulus of concrete slab.

Column stiffness $K_c$ is independent of $K_t$ and is calculated conventionally, using actual column moment of inertia between the slabs and an infinite moment of inertia within the slabs.

Slab stiffness $K_s$ is calculated conventionally with the full transverse span ($l_2$). ACI 318 (2002) also recommended that the effects of column capitals and drop panels be included in the model by increasing the moment of inertia of that portion between the center of the column and the face of the column, bracket, or capital by the factor $1/(1-c_2/l_2)^2$. This increase is to account for the increased flexural stiffness of the slab-column connection region.

The explicit transverse torsional member model has several advantages. Structural modeling is simple and direct, requiring very few hand computations. Also, computed member actions in the slab-beams and transverse torsional members can be used directly for design of slabs and spandrels, respectively. Finally, this model can be developed even for the true three-dimensional analysis of slab system under combined gravity and lateral loads. Two sets of equivalent frames, each running parallel to one of the building's two principal plan orientations, can be combined to form a single three-dimensional model. This single model can be used to calculate actions in all members (slabs, columns, and spandrels) under as many combinations of gravity and lateral loads as desired.

### Masonry infill walls

Masonry infill walls are typically used in reinforced concrete buildings and are considered by engineers as nonstructural components. Even if they are relatively weak when compared with structural components, they can drastically alter the response of structure. The presence of masonry infill walls can modify lateral stiffness, strength, and ductility of structure. For these reasons, in this study, masonry infill walls are considered in the evaluation of seismic capacity. Masonry infill walls are modeled using equivalent strut concept based on recommendations of FEMA-273 (1997). More detail is given by Kiattivisanchai (2001) and Imarb (2002).

### Foundation model

Behavior of foundation components and effects of soil-structure interaction are investigated in this study. Soil-structure interaction can lead to modification of building response. Soil flexibility results in period elongation and damping increase. The main relevant impacts are to modify the overall lateral displacement and to provide additional flexibility at the base level that may relieve inelastic deformation demands in the superstructure.

Most buildings in Bangkok are constructed by using deep foundations (pile foundations). In this study, Winkler component model (Figure 4), which can be represented by a series of independent or uncoupled lateral and axial springs simulating soil-pile interaction, is used in order to model the behavior of foundations. By using this model, the load-deformation relations of vertical and horizontal geotechnical components can be presented.

Apart from the load-deformation relations of vertical geotechnical components, under earthquake loading, the load-deformation relations of lateral geotechnical components are also important. The analysis of a pile under lateral loading is complicated by the fact that the soil reaction depends on the pile movement, and the pile movement is dependent on the soil response. In this study, the

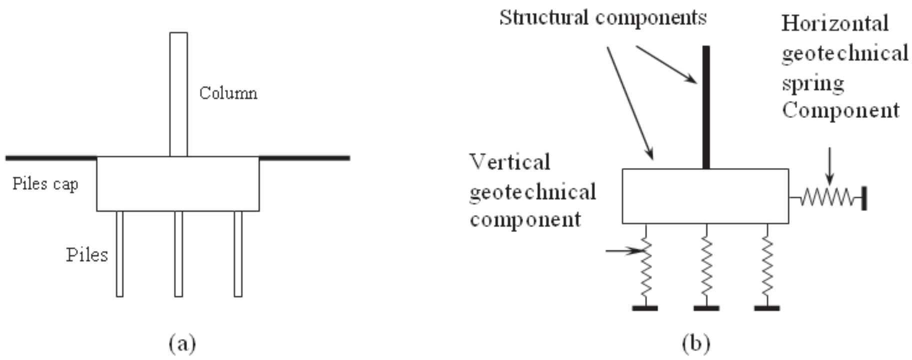



**Figure 4. Winkler component model, (a) deep foundation, (b) model for analysis**

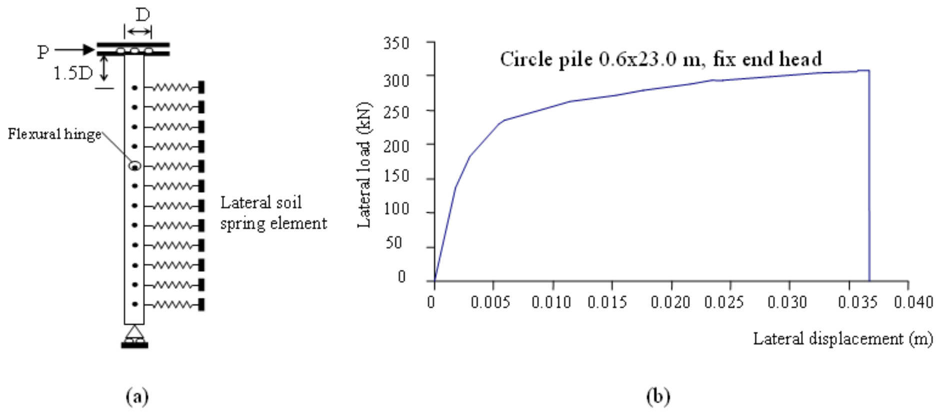



**Figure 5. (a) Refined pile model with fixed head, (b) lateral load-displacement relationship of driven pile**

subgrade-reaction model, which was originally proposed by Winkler in 1867, is used to determine the lateral force-deformation relations. By using this model, soil is replaced by a series of independent spring elements. The force-deformation relation of soil spring element is approximated by an elastic-perfectly plastic model that has an initial stiffness equal to horizontal modulus of subgrade reaction, and the maximum force equal to the ultimate soil resistance.

Based on the subgrade-reaction model and the above assumptions, a pile is modeled as shown in Figure 5a Moreover, the flexural hinge having moment-rotation relation is introduced in this model, along the pile length, to represent the flexural behavior of reinforced concrete pile under lateral load. The predicted lateral load-displacement of pile shown in Figure 5b is in good agreement with the test results obtained from static lateral load test of three sites in Bangkok. Therefore, the above approach is used to obtain the lateral-displacement at the pile top in this study. More detail is given by Kiattivisanchai (2001).

## Numerical example 1

### Building descriptions

The building in this example is a typical flat-plate building in Bangkok. It is a nine-storey post-tensioned flat-plate building. It has three spans in the N-S direction and 8 spans in the E-W direction. The storey height is 2.6 meters with a total height of 23.4 meters. The building is rectangular in plan, 14.40 meters by 36 meters. Gravity loads include dead loads and live loads. The structural system consists of post-tensioned flat-plate thickness of 20 cm supported by reinforced concrete columns of dimensions 30 cm by 60 cm and 40 cm by 80 cm for exterior and interior connections, respectively. The spans are 4 meters for exterior spans and 6 meters for interior spans while the transverse span is 6 meters. The lateral resistance system is provided by reinforced concrete flat-slab frame with the contribution of brick infill walls.

In the foundation system, each column rests on pile cap supported by a group of driven piles with 5 piles for interior column and 3 piles for exterior column. Each pile is of circular shape 0.6 meters in diameter and 23 meters in length. It is designed for a vertical safe load of 80 tons. The cylinder compressive strengths of concrete column and post-tension slab are 23.5 MPa and 32 MPa, respectively. The expected yield strength of steel bars and prestressing steels are 460 MPa and 1,670 MPa, respectively, including the overall strength factors of steel bar. More detail of the building is given by Intaboot (2003) and Tam (2003).

## Seismic capacity curves

The capacity curves are obtained from nonlinear static pushover analysis. The capacity curves are in the form of normalized base shear coefficient versus roof drift ratio. Each capacity curve represents the capacity of the building in each case study. The curve represents the yielding and failure mechanism of each component in the structure. In order to explain the response of the structure through the capacity curve, analysis results are described by a set of characters: xx-xxx(x-x). The first two characters represent the damage type of building element, such as, flexural yielding, flexural failure or punching failure. The next group of characters indicates the name of the element and the number in parenthesis indicates the position of the element. For example, PF-IC(3-5) means "punching failure of interior connection at the third to fifth floor". In addition, the damage distribution patterns of the structure are presented.

The evaluation is conducted with realistic conditions taking into account cracking in members, foundation flexibility system, and masonry infill wall. In order to investigate the effects of foundation, a frame with flexible base and a frame with fixed base are analyzed. Frame with and without infill walls are also analyzed.

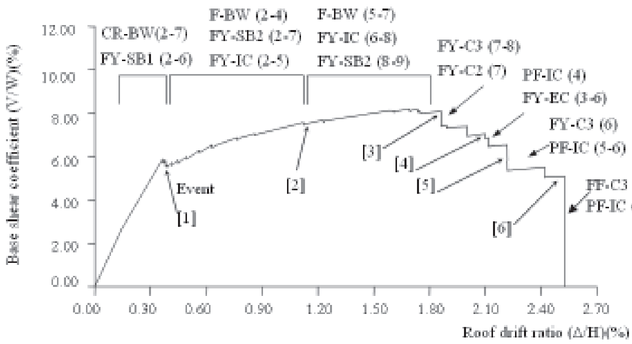
### *Behavior of slab-column frame building*

The capacity curve of the flat-plate building in terms of normalized base shear and roof drift is shown in Figure 6. The failure mechanisms start from the cracking of masonry infill walls (IF) at the 2$^{nd}$ to 7$^{th}$ floors. Then, flexural yielding of slab-beams at the 2$^{nd}$ to 6$^{th}$ floors occur resulting in significant decrease of the stiffness of the building. After that, interior slab-column connections yield at the 2$^{nd}$ to 8$^{th}$ floors, interior (C3) and exterior (C2) columns yield at the 7$^{th}$ to 8$^{th}$ floors, and then the lateral capacity of the building begins to decrease. Punching failures are also observed at the interior connections at the 4$^{th}$ to 7$^{th}$ floors. Finally, the lateral capacity drops immediately when flexural failures occur at the column of the 7$^{th}$ floor. It should be noted that the flat-plate building clearly behaves like the strong column-weak beam mechanism. In the evaluation of the flat-plate building, the maximum base shear coefficient is about 8% while the maximum roof drift ratio is 2.5%.

Figure 7 shows the local drift profiles for different levels of roof drift. The first profile is at the first yield point of the structure corresponding to 0.4% of roof drift. The final profile is before the structure collapses, at 2.5 % of roof drift. It can be seen that the maximum local drift occurs at the 4$^{th}$ floor. This should be the case since the first yield of interior connections occurs at the 4$^{th}$ floor, and failure develops widely from the 3$^{rd}$ to 7$^{th}$ floors where the drifts are larger than that of other floors.

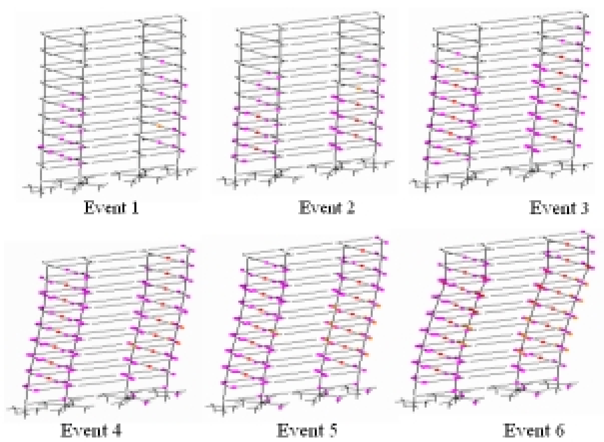
### *Effects of infill wall on capacity*

Figure 8 shows the comparison between capacity curves of frames with and without infill walls. As seen in the figure, masonry infill walls increase the lateral initial stiffness of frame significantly. However, the lateral capacity does not increase much because the infill walls are quite weak in comparison with structural components.




F = Failure, CR = Cracking, BW = Brick wall
FY = Flexural yielding, FF = Flexural Failure
PF = Punching Failure, EC = Exterior slab-column connection
IC = Interior slab-column connection

**(a)**




**(b)**

**Figure 6. (a) Capacity curve, and (b) sequence of yielding and failure of slab-column frame**

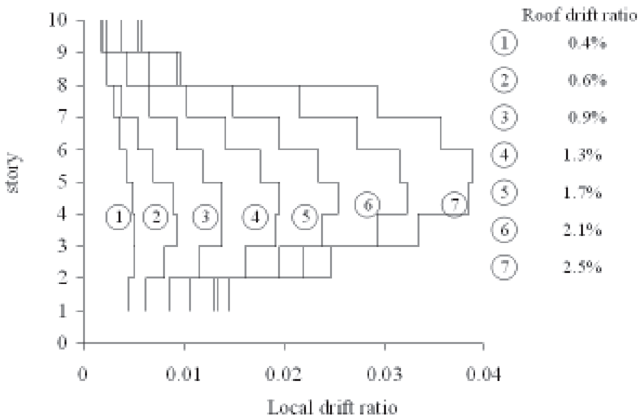



**Figure 7. Local drift profiles for different levels of roof drift**

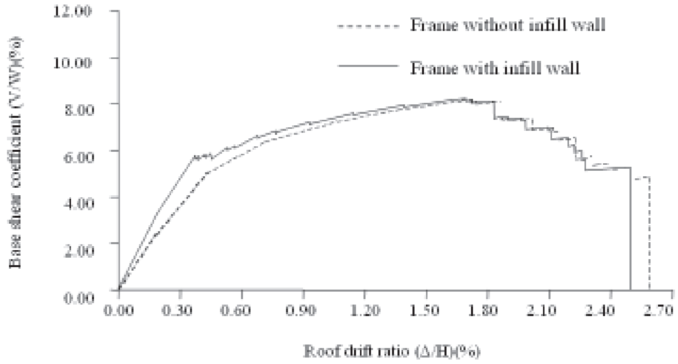



**Figure 8. Effects of infill wall on capacity**

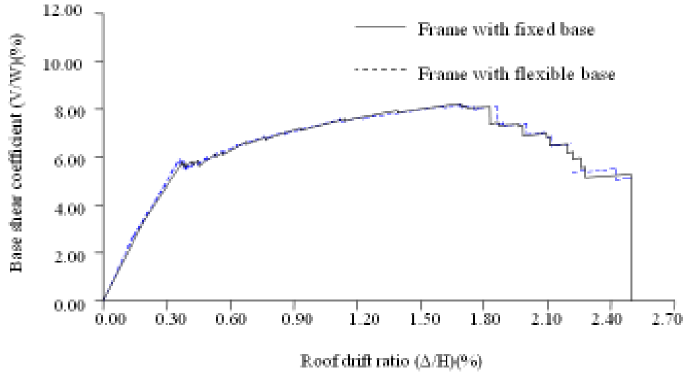



**Figure 9. Effects of foundation on capacity**

Therefore, they fail early before other structural components. After the failure of the infill walls, the capacity curves of both cases are nearly the same.

***Effects of foundation on capacity***

The effects of foundation stiffness on the capacity of the buildings are also evaluated. The capacity curves for flexible base and fixed base buildings are plotted in Figure 9. Slight difference between the two cases is observed. At the same load level, the roof displacement of flexible support is slightly higher than that of fixed support. This is because the flexible support allows the building to rotate and translate resulting in additional displacement at the roof. However, for this building, the pile foundation are relatively stiff and do not significantly affect the building capacity and response.

**Performance evaluation of building**

***Effects of ground motion types***

Elastic and inelastic response spectra for scaled ground motion of El Centro and simulated ground motion for Bangkok for 500-year return period are shown in Figure 10 for buildings with 5% damping and ductility ratio (μ) ranging from 1-4. It is can be seen that buildings with natural period between 0.8-1.2 second (about 8-12 stories high) have large response under simulated ground motion of Bangkok.

***Effects of intensity of earthquake on response***

The comparison of capacity and demand spectrum in ADRS format for different levels of intensity of earthquake ground motions corresponding to different return periods are shown in Figure 11 for two types of ground motions. As

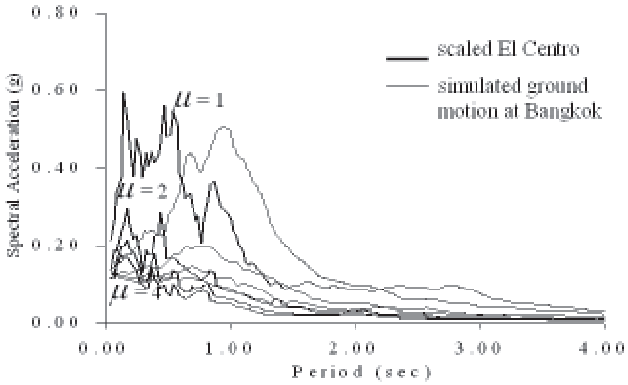
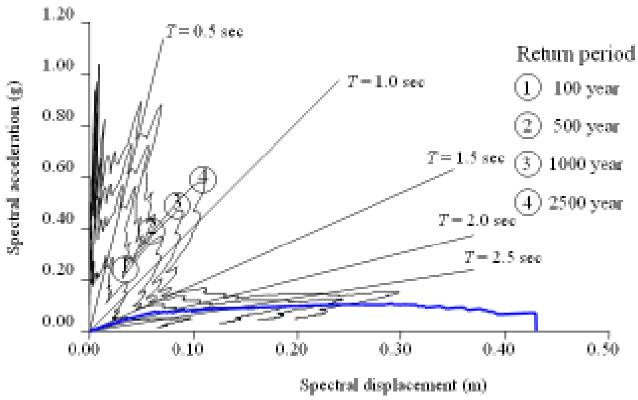
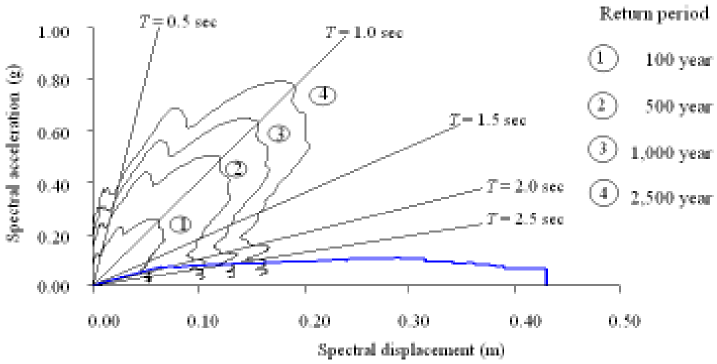



**Figure 10. Elastic and inelastic response spectra for scaled ground motions of El Centro and simulated ground motion of Bangkok for a 500-year return period for ductility ratio ranging from 1-4**




**(a)**




**(b)**

**Figure 11. Comparison of capacity and demand spectrum for different levels of intensity of earthquake ground motions for ductility ratio of 1**
**(a) under scaled ground motion of El Centro,**
**(b) under simulate ground motion of Bangkok**

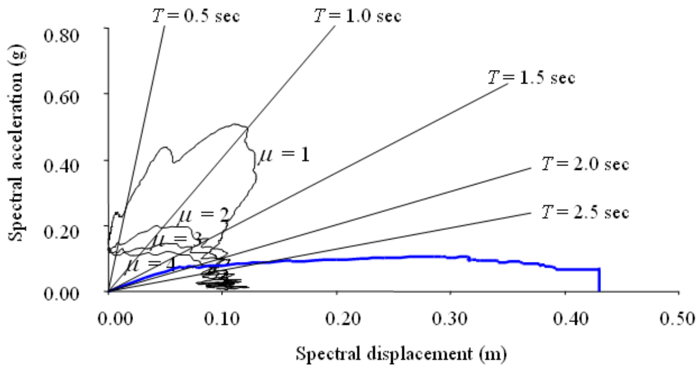



**Figure 12. Comparison of capacity and demand spectrum for simulated ground motion of Bangkok for a 500-year return period for ductility ratio ranging from 1-4**

seen in Figures 10 and 11, for this building with a natural period of 1.7 second, no significant difference in building response between the two types of ground motion is observed. The building responds within the elastic range when subjected to earthquake ground motions of 50% (100-year return period) probability of exceeding in a 50-year exposure period. For ground motions of 10% (500-year return period), 5% (1,000-year return period), and 2% (2,500-year return period) probability of exceedance in a 50-year exposure period, the building deforms beyond the elastic range resulting in failure of masonry infill wall, flexural yielding of slab-beams, and yielding of slab-column connections. The detail of yielding can be observed by comparing the performance point shown in Figure 11 with the capacity curve and failure mechanism in Figure 6. Finally, this the building will not collapse when subjected to the highest intensity earthquake ground motions expected in Bangkok despite the fact that it was designed without any consideration for seismic loading.

***Effects of ductility ratio on response***

Comparison of capacity and demand spectrum for simulated ground motion for Bangkok for a 500-year return period is shown in Figure 12 for ductility ratio ranging from 1-4. It is can be seen that for ductility ratio greater than 1, the building response is moderately reduced.

**System strengthening and stiffening**

The method is limited not only to the existing and new buildings but also for the retrofit of inadequate buildings. In this section, the method is used to simulate the response of buildings to improve the seismic performance, both lateral strength and ductility of the selected building. Two strengthening schemes are presented based on the results obtained from pushover analysis of the example building.

***Drop panel***

Drop panels of 15 cm additional thickness below the floor are added to the building. The effect of the drop panels on seismic capacity is shown in Figure 13. The result shows that the lateral capacity of the building increases about 18%. Thus the drop panels increase the strength and stiffness of the building significantly.

***Shear wall***

Shear wall of height 23.4 meters, width 2.5 meters and thickness 0.3 meters is added to the building. The resulting seismic capacity is shown in Figure 14. The result shows that the lateral capacity of the building increases about 40%. Thus the shear wall increases the strength and stiffness of the slab-column building significantly.

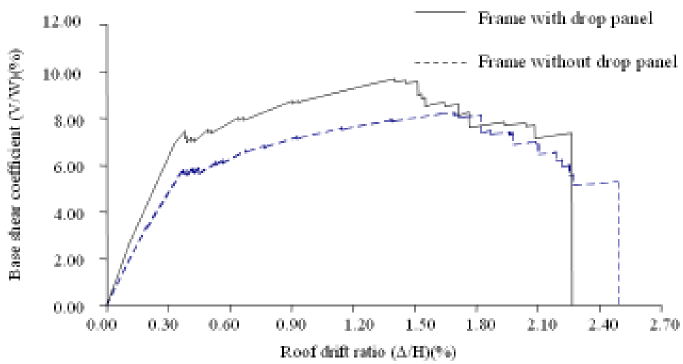



**Figure 13. Effect of drop panel on seismic capacity**

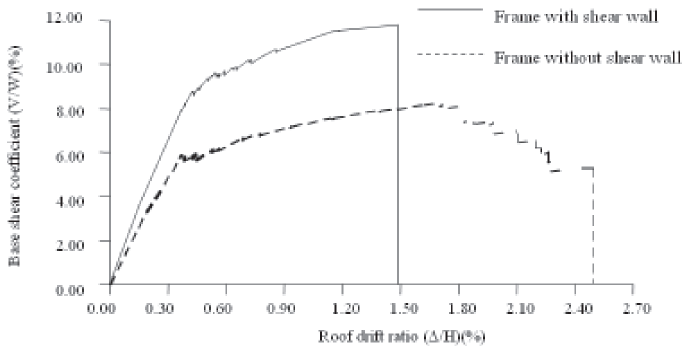



**Figure 14. Effect of shear wall on seismic capacity**

## Numerical example 2

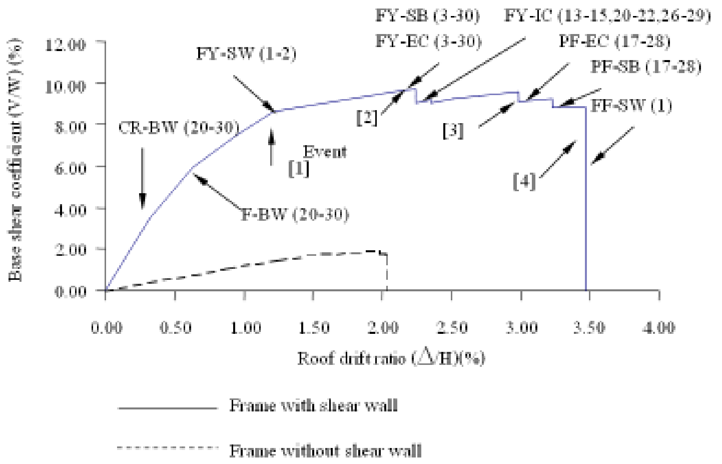
### Building descriptions

The building in this example is a typical flat-plate office building in Bangkok. It is a 30-storey post-tensioned flat-plate building. It has three spans in the N-S and E-W directions. The storey height is 2.7 meters for the first 10 stories and 3.7 meters for other stories with a total height of 95.25 meters. The building is rectangular in plan, 26.9 meters by 32.8 meters. Gravity loads include dead loads and live loads. The structural system consists of post-tensioned flat-plate thickness of 25 cm supported by reinforced concrete columns and shear wall. The spans are 9 meters for exterior and interior spans. The lateral resistance system is provided by reinforced concrete shear wall with the contribution of brick infill walls.

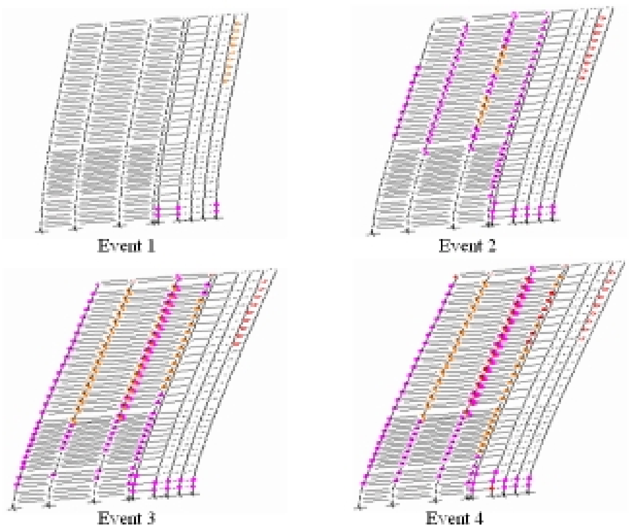
In the foundation system, the columns are supported by 3 meter thickness of mat foundation width. Each pile is of circular shape 1.5 meters in diameter and 60 meters in length. It is designed for a vertical safe load of 360 tons. The cylinder compressive strengths of concrete column and post-tension slab are 23.5 Mpa. The expected yield strength of steel bars and prestressing steels are 460 MPa and 1,670 MPa, respectively, including the overall strength factors of steel bar. More detail of the building is given by Intaboot (2003).

### Seismic capacity curves

The capacity curve of the flat-plate building in terms of normalized base shear and roof drift is shown in Figure 15. The failure mechanisms start from the cracking of masonry infill walls (IF) at the 20$^{th}$ to 30$^{th}$ floors. Then, flexural yielding of shear wall at the 1$^{st}$ to 2$^{nd}$ floors occur resulting in significant decrease of the stiffness of the build-




**(a)**




**(b)**

**Figure 15. (a) Capacity curve, and (b) sequence of yielding and failure of slab-column frame**

ing. After that, interior slab-column connections yield at the $4^{th}$ to $30^{th}$ floors, interior and exterior columns yield at the $3^{rd}$ to $30^{th}$ floors, then the lateral capacity of the building begins to decrease, and then the slab-column connections and exterior columns fail at the $17^{th}$ to $28^{th}$ floors. Finally, the lateral capacity drops abruptly when flexural failures occur at the shear wall at the 1st floor. It should be noted that the flat-plate building clearly behaves like the strong column-weak beam mechanism. In the evaluation of the flat-plate building, the maximum base shear coefficient is about 9.5% while the maximum roof drift ratio is 3.45%.

**Performance evaluation of building**

The comparison of capacity and demand spectrum in ADRS format for different levels of intensity of earthquake ground motions in Bangkok are shown in Figure 16. As seen in this figure, the building responds within the elastic range when

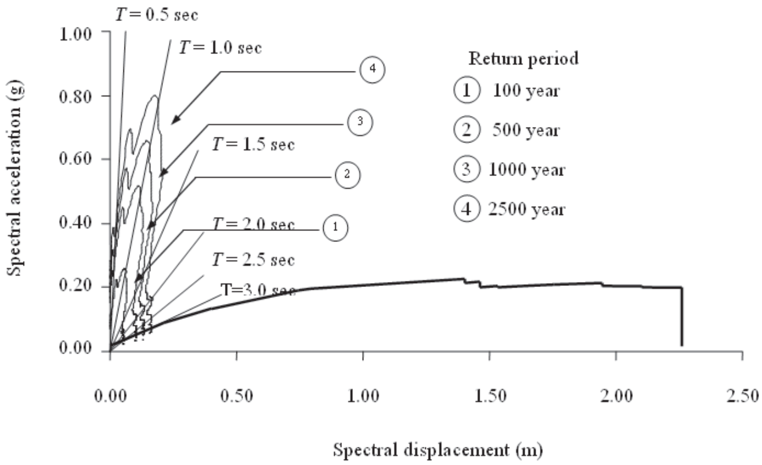



**Figure 16. Comparison of capacity and demand spectrum for different levels of intensity of simulated earthquake ground motions of Bangkok for ductility ratio of 1**

subjected to earthquake ground motions of 50% (100-year return period), 10% (500-year return period), 5% (1,000 year return period), and 2% (2,500-year return period) probability of exceedance in a 50-year exposure period. It should be noted that the building responds within elastic range and will not collapse when subjected to the highest intensity earthquake ground motions expected in Bangkok despite the fact that the building was designed without consideration for seismic loading. This is because it was designed for wind load, and the base shear under the design wind load is about two times that of the design earthquake load (Intaboot, 2003). However, since the effects of higher vibration modes on building response were not considered in this study, a further investigation should be studied.

## Conclusions

Seismic capacity evaluation of post-tensioned concrete slab-column frame buildings designed only for gravity loads and wind load is presented. The series of nonlinear pushover analyses are carried out by using the computer program SAP 2000. An equivalent frame model with explicit transverse torsional members is introduced for modeling slab-column connections. From the numerical examples of the 9-and 30-storey post-tension flat-plate buildings in Bangkok, the following conclusion can be drawn.

1. Sequence of yielding and failure of slab-column frame can be clearly seen by the nonlinear pushover analysis. The flat-plate building clearly behaves like the strong column-weak beam mechanism. The maximum base shear coefficients for 9- and 30-storey buildings are about 8% and 9.5% while the maximum roof drift ratios are 2.5% and 3.45%, respectively.

2. For ground motions of 10% (500-year return period) probability of exceedance in a 50-year exposure period, the 9-storey building deforms beyond the elastic range which leads to failure of masonry infill wall, flexural yielding of slab-beams, and yielding of slab-column connections. However, the building will not collapse when subjected to the highest intensity earthquake ground motions expected in Bangkok despite the fact that the building was designed without any consideration for seismic loading.

3. The 30-storey building responds within the elastic range when subjected to the highest intensity earthquake ground motions expected in Bangkok. This is because it was designed for wind load, and the base shear under the design wind load is about two times that of the design earthquake

load. However, since the effects of higher vibration modes on building response were not considered in this study, a further investigation should be studied.

4. The buildings with natural period between 0.8-1.2 second (about 8-12 stories) have large response under simulated ground motion at Bangkok.

5. Masonry infill walls increase the lateral initial stiffness of frame significantly. However, the lateral capacity does not increase much because the infill walls are quite weak in comparison with structural components.

6. The pile foundations for these buildings are relatively stiff and do not significantly affect the building capacity and response.

7. The system strengthening and stiffness can significantly improve the seismic performance of the slab-column frame building. In this study, the lateral capacity of the 9-storey building can be increased about 18% and 40% by adding drop panels and shear wall, respectively.

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