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SEISMIC CAPACITY EVALUATION OF POST-TENSIONED CONCRETE SLAB-COLUMN FRAME BUILDINGS BY PUSHOVER ANALYSIS

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SUMMARY

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 analysis 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 at Bangkok site and then the seismic performance can be evaluated. The numerical examples are performed on the 9-story post-tension flat-plate building in Bangkok. The results show that in general post-tensioned concrete slab-column frame buildings 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.

INTRODUCTION

Bangkok, the capital city 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 implementation of seismic load in North of Thailand in 1997 and at present, many existing post-tensioned concrete slab-column buildings in Bangkok may have been designed without consideration of seismic loading. Therefore, the evaluation of seismic capacity of existing buildings in Bangkok is need. The static pushover procedure has been

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presented and developed over the past twenty year [1-6]. The method is also described and recommended as a tool for design and assessment purposes by the National Earthquake Hazard Reduction Program 'NEHRP' (FEMA 273) [7] guidelines for seismic rehabilitation of existing buildings and by the Applied Technology Council (ATC-40) [8] guidelines for seismic evaluation an retrofit of Concrete building. Moreover, the technique is accepted by the Structural Engineers Association of California 'SEAOC'(Vision 2000) [9] among other analysis procedure with various level of complexity. This analysis procedure is selected for its applicability to performance-based seismic design approaches and can be use 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 [1-6], 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 analysis are carried out by using the computer program SAP2000. A modified equivalent frame model with explicit transverse torsion members is introduced for modeling slab-column connections.

SEISMIC CAPACITY EVALUATION METHOD

Pushover Analysis

The nonlinear static pushover analysis is a sample option for estimating the strength capacity 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 [10] which is a three-dimensional static and dynamic finite element analysis and design of structure program allows for strength and stiffness degradation in the components by providing the force-deformation criteria for hinges used in pushover analysis and 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 depending on the type of component, failure mechanism, ratio of reinforcement and many parameters which are described in the ATC-40 [8] and FEMA-273 [7].

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 so-called Acceleration-Displacement Response Spectra (ADRS)

To construct the capacity spectrum, capacity curve of the multistory building are converted into the capacity curve of the equivalent single degree of freedom (SDOF) systems base on the capacity curve, which in term of base shear and lateral roof displacement, obtain from pushover analysis (Fig.1a). Any point, V and Δ_{roof}, on the capacity curve is converted to the corresponding point S_a and S_d on the capacity spectrum using the equation

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

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

where S_a = structural acceleration; S_d = structural displacement; PF_1 = modal participation factor for the first natural mode; α_1 = modal mass coefficient for the first natural mode; V = base shear; W = building dead weight plus likely live lodes; Δ_{roof} = lateral roof displacement; $\phi_{1,roof}$ =amplitude of the first natural mode at roof.

In this study, the constant ductility yield strength demand spectrums are constructed base on a lump mass SDOF system. To compare with the capacity spectrum, every point on a demand spectrum will be transformed from the standard S_a versus T format to ADRS format which can be done by using equation

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

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 the relationship as follow:

$$T = 2\pi\sqrt{\frac{S_d}{S_a g \mu}} \tag{4}$$

The demand spectra both standard format and ADRS format are presented in (Fig.1b). The intersection between capacity and demand spectrum having the same ductility factor, μ, is the performance point (Fig.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 formats. 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 specified displacement ductility ratio. The displacement ductility 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.

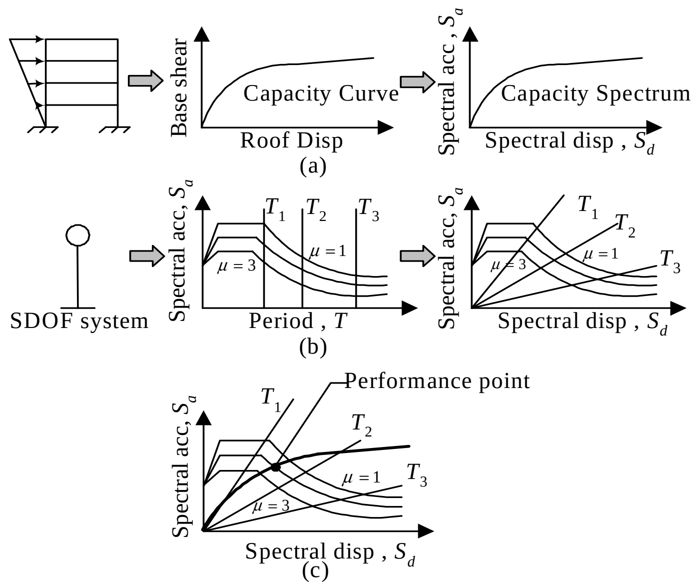


Fig.1. (a) capacity spectrum, (b) demand spectrum, and (c) capacity spectrum superimposed over demand spectrum in ADRS format

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. The seven strong ground motion records are selected from actual seismograms from far-field sites. These ground motions are use as input rock outcrop earthquake motions and they are scaled to the require intensity. In addition, the amplification effect on Bangkok's soft soils is also included to simulate the ground motions in Bangkok. The detail is given by Wanichai et al.[11]. Fig.2 shows simulated ground motion at Bangkok site in 500 year return period

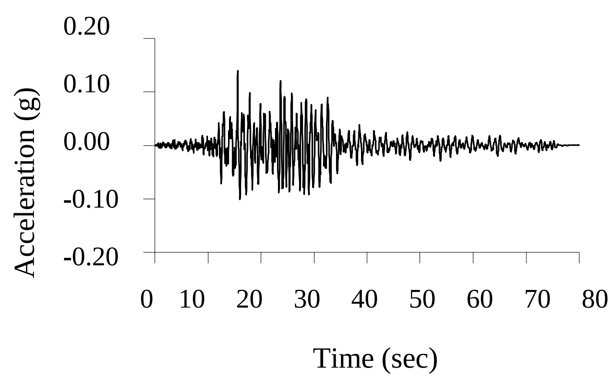


Fig.2. Simulated ground motion at Bangkok site in 500 year return period

BUILDING MODEL

Model of Slab-Column Frame

An improvement of analyzing reinforced-concrete slab-column frame model, termed the explicit transverse torsional member method was proposed by Cano and Klingner [12] and applied in this study. Model of slab-column frame is shown in Fig.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, made using explicit transverse torsional members, permits the modeling of moment leakage as well as slab torsional flexibility. While the resulting frame is nonplanar, 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.

$$K_t = \sum \frac{9E_{cs}C_t}{l_2\left(1-\frac{c_2}{l_2}\right)^3} \tag{5}$$

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 the arbitrary length L for the torsional members, the torsional stiffness J is then calculated by expression

$$J = K_t L / G \tag{6}$$

where G is the shear modulus of slab concrete.

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 [13] also recommended that the effects of column capitals and drop panels can be included in the model by increasing the moment of inertia of that portion between the center of the column and face of column, bracket, or capital by the factor $1/(1-c_2/l_2)^2$. This increasing is to account for the increased flexure 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 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. By these reasons, in this study, masonry infill walls are considered in the evaluation of seismic capacity. In this study, masonry infill walls are modeled using equivalent strut concept based on recommendations of FEMA-273 [7].The more detail is given by Kiattivisanchai [5] and Imarb [6].

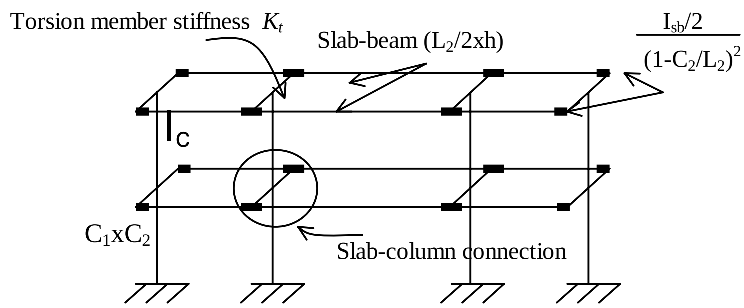


Fig.3. Model of slab-column frame building

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 superstructure.

Most buildings in Bangkok are constructed by using deep foundations (pile foundations). In this study, Winkler component model (Fig.4), which represents by 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 are presented.

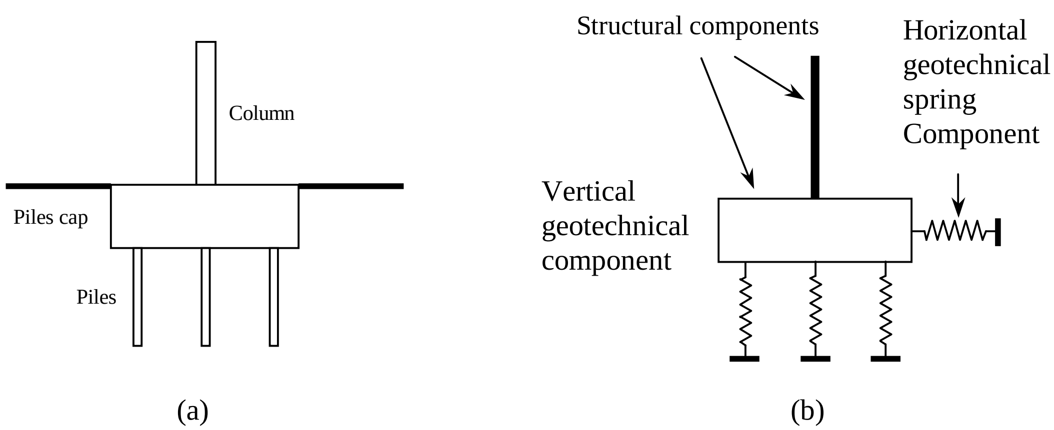


Fig.4. Winkler component model, (a) deep foundation, (b) model for analysis

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 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 the series of independent spring elements. The force deformation relation of soil spring element is approximated by an elastic perfectly plastic model that has the 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, pile is modeled as shown in Fig. 5b. 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 Fig. 5b is in good correlation with the test results obtained form static lateral load test of three site in Bangkok. Therefore, the above approach is used to obtained the lateral-displacement at the pile top in this study. The more detail is given by Kiattivisanchai [5].

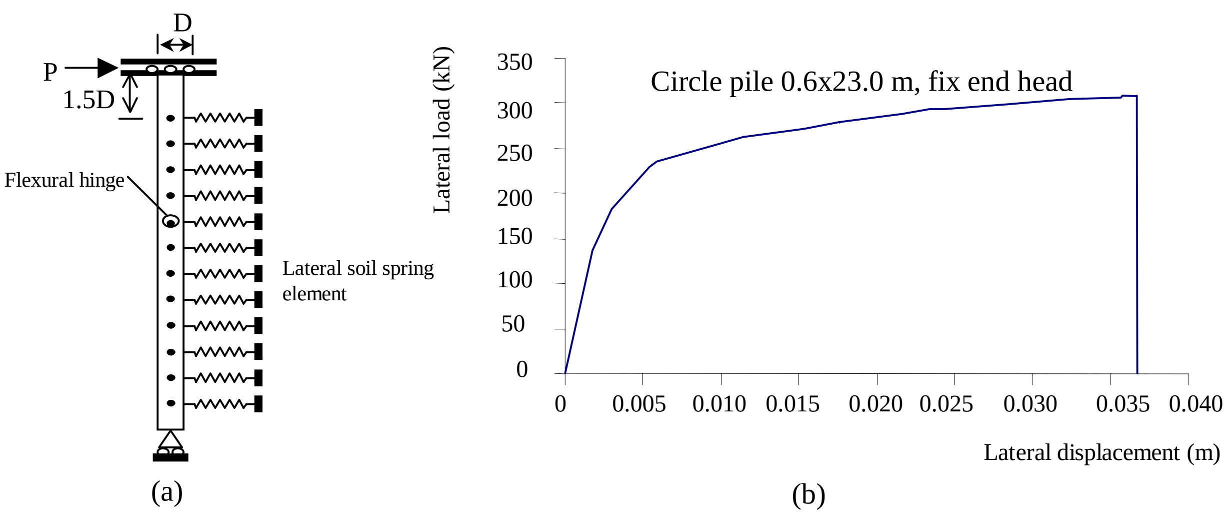


Fig.5. (a) Refined pile model with fixed head, (b) lateral load-displacement relationship of driven pile

NUMERICAL EXAMPLES

Building Descriptions

The building in this study is a typical flat-plate building in Bangkok. It is a nine-story post-tensioned flat-plate building. It has three spans in the N-S direction and 8 spans in the E-W direction. The story height is 2.6 meters with the total height of 23.4 meters. The building is rectangular in plan, 14.40 meters by 36 meters.

The gravity loads including dead loads and live loads are carried by structural system composed of post-tensioned flat-plate thickness 20cm supported by reinforce concrete columns which are dimension of 30cm by 60cm and 40cm by 80cm for exterior and interior connections, respectively. The span is 4 meters for exterior span and 6 meters for interior span while the transverse span is 6 meters. In addition, the lateral resistance system is provided by reinforce concrete flat-slab frame system with the contribution of brick infill walls.

In the foundation system, each column is comprised pile cap supported by a group of cast in place reinforce concrete cast insitu driven pile with 5 piles for interior column and 3 piles for exterior column. Each pile is 0.6 meters in diameter of circular shape and 23 meters in length. The pile is designed for 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.

Seismic Capacity Curves

The capacity curves can be 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 structure through the pushover curve, a general format is introduced. It is in the form of xx-xxx(x-x). The two first characters represent the damaged type of building element, such as, flexural yielding, flexural failure or punching failure. The second character group is for the name and position of the component. 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 represented.

The evaluation is conducted with the most realistic condition where all the effects of cracking, the foundation system, and masonry infill wall are included. In order to investigate the effects of foundation modeling, a frame with flexible base and a frame with fixed base are analyzed. A frame is also considered with and without infill walls modeling.

Behavior of slab-column frame building

The capacity curve in term of normalized base shear and roof drift of the flat-plate building is shown in Fig. 6 The failure mechanisms start from the cracking of masonry infill walls (IF) at the 2nd to 7th floors. Then, flexural yielding of slab-beams at the 2nd to 6th floors occur and result in significant decreasing of the stiffness of the building. After that, interior slab-column connections yield at the 2nd to 8th floors, interior (C3) and exterior (C2) columns yield at the 7th to 8th floors, and then the lateral capacity of the building begins to decrease. The punching failures are also observed at the interior connections at the 4th to 7th floors. Finally, the lateral capacity drops immediately when the flexural failures occurred at the column at the 7th floor. It is should be note that the flat-plate building clearly behave 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%.

Fig.7 shows local drift profiles in building for difference level of roof drift. First stage is at the first yield point of the structure corresponding to 0.4% of roof drift. The final stage is before the structure collapses, at 2.5 % of roof drift. It can be seen from the graph that the maximum local drift occurs at the 4th floor. This result well fits that the first yield of interior connection at the 4th floor is observed, and the failure mechanisms develop widely from the 3rd to 7th floor where the drifts are larger than that of other stories.

Fig. 8 shows the comparison between capacity curves of frames with and without infill walls modeling. As observed from the graph, 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. Therefore, they fail early before failing of structural components. After the failure of the infill walls, the capacity curves of both cases are nearly the same.

Effects of infill wall on capacity

Fig. 8 shows the comparison between capacity curves of frames with and without infill walls modeling. As observed from the graph, 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. Therefore, they fail early before failing of structural components. After the failure of the infill walls, the capacity curves of both cases are nearly the same.

Effects of foundation on capacity

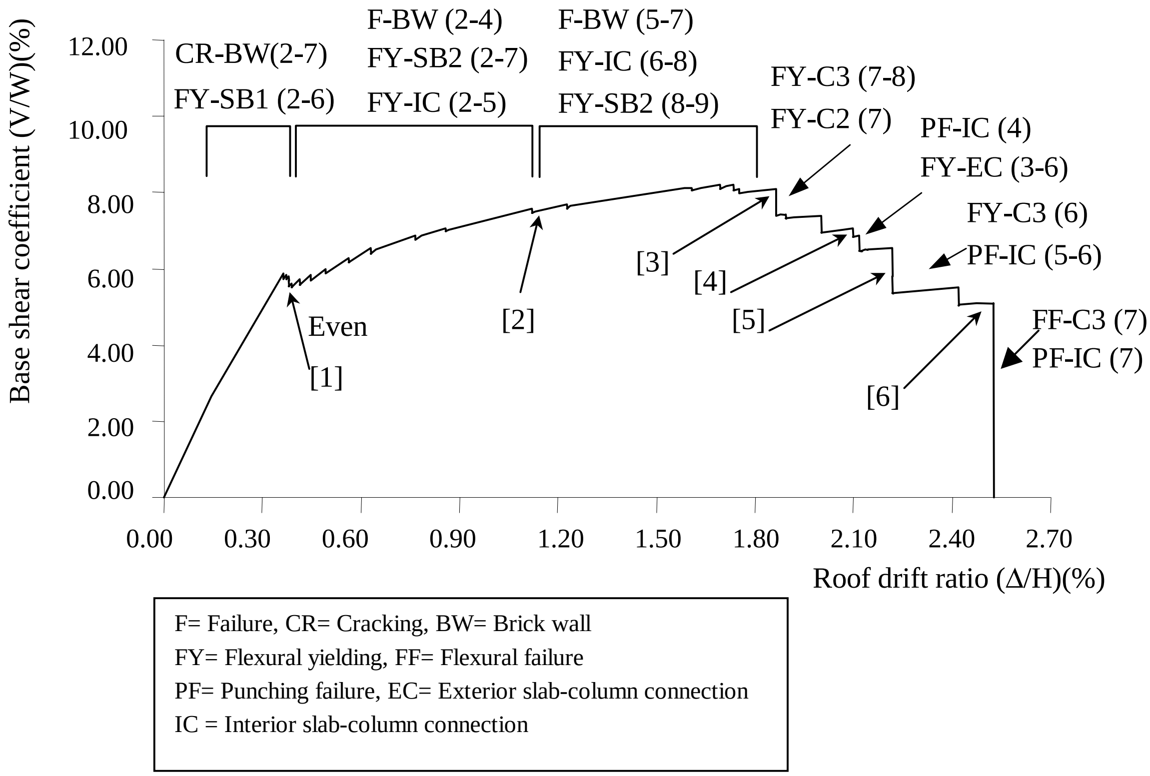
The effects of foundation modeling on the capacity of the buildings are also considered. The capacity curves obtained from flexible base modeling and fixed base are plotted in Fig. 9. Slightly difference between the behavior of two cases is observed. At the same load level, the roof displacement of flexible support is slightly higher than that of fixed support. This is because the flexible support allows the building to rotate and translate that result in additional displacement at the roof. However, in this building, the pile foundation are relatively stable and do not significantly affect the building capacity and response.

Performance Evaluation of Building

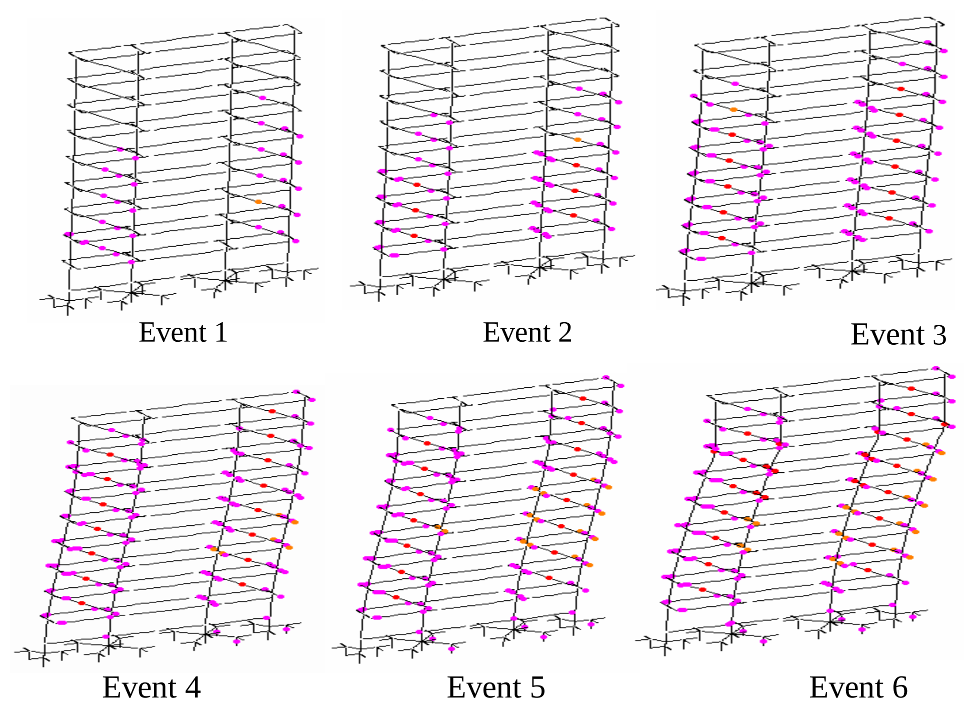
The comparison of capacity and demand spectrum in ADRS format for different levels of intensity of earthquake ground motions which have possible to occur in Bangkok are show in Fig 10 and 11. As observe from both figures, the building responds within elastic range when it subjected to earthquake ground 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 exceeding in a 50-year exposure period, the building deforms into inelastic range which leads to 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 Fig 10 and 11 with the pushover curve and failure mechanism in Fig 6. Finally, this building will not collapse when it is subject to the highest intensity earthquake ground motions expected in Bangkok despite the fact that the building was designed without any consideration on seismic loading.

System Strengthening and Stiffening

The evaluation method is not limited only for evaluation of existing structures but also for design the new structures and for finding proper retrofit schemes in improving the performance of structures. Therefore, in this section, the evaluation method is used to find and appropriate way to improve the seismic performance, both lateral strength and ductility, of the selected building. For this reason, the two strengthen schemes are presented based on the results obtain from pushover analysis of the example building.



(a)



(b)

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

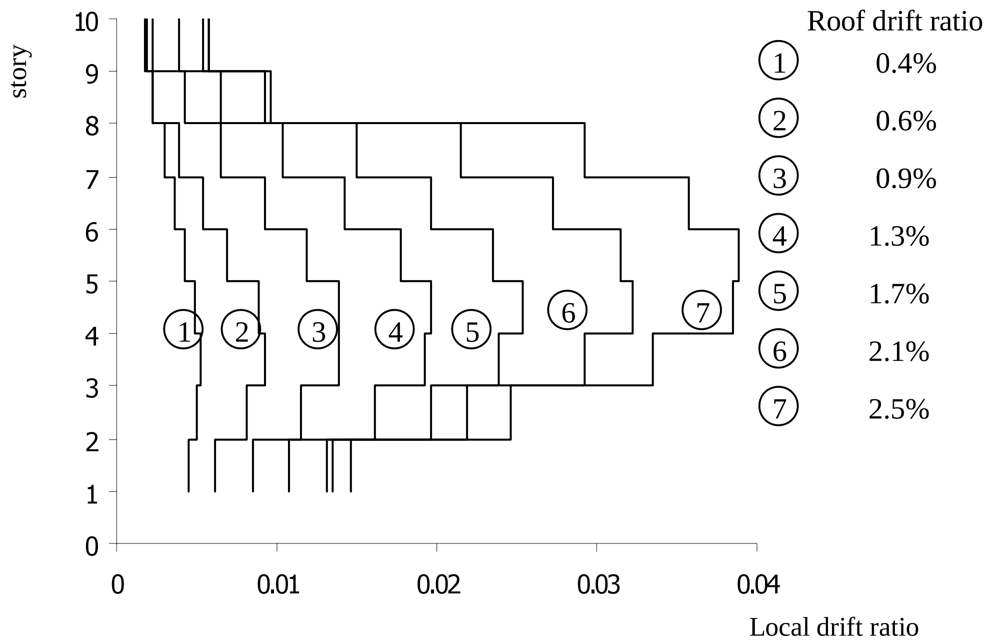


Fig.7. Local drift profiles in building for difference level of roof drift

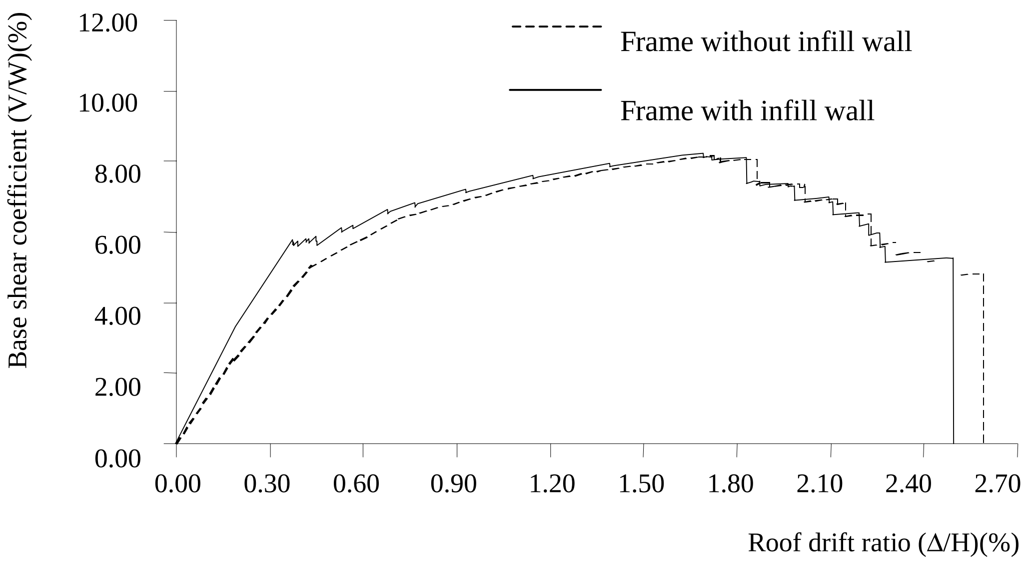


Fig.8. Effects of infill wall on capacity

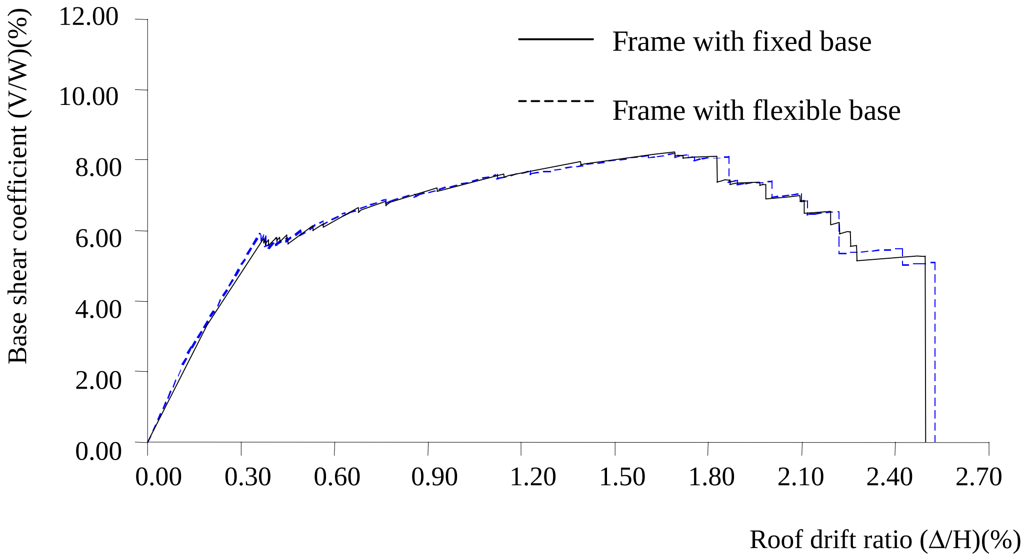


Fig.9. Effects of foundation on capacity

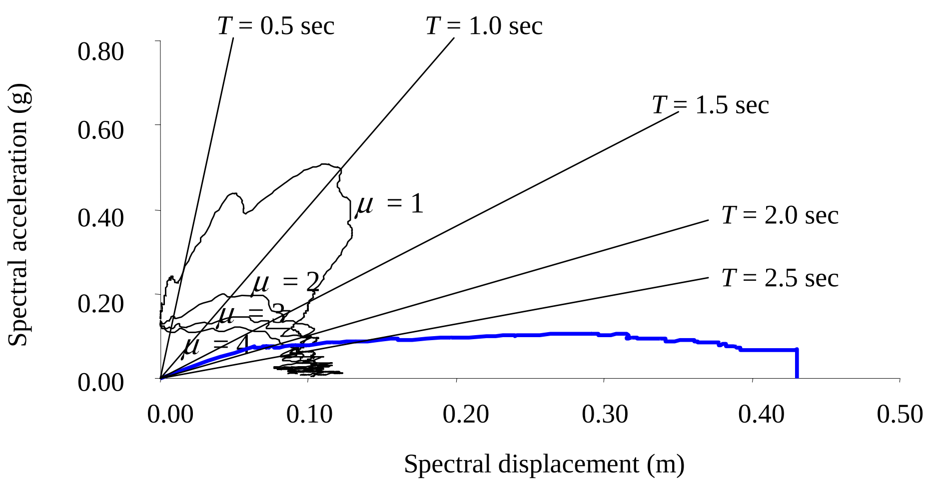


Fig.10. Comparison of capacity and demand spectrum for 500 year return period
for ductility ratio equal 1-4.

Drop panel

Drop panel with additional 15 cm of thickness below the floor is introduced to the building. The effect of the drop panel on seismic capacity is shown in Fig. 12. The results show that the lateral capacity of the building increase about 18 %. It is can be seen that the drop panel can increase the strength and stiffness of the building significantly.

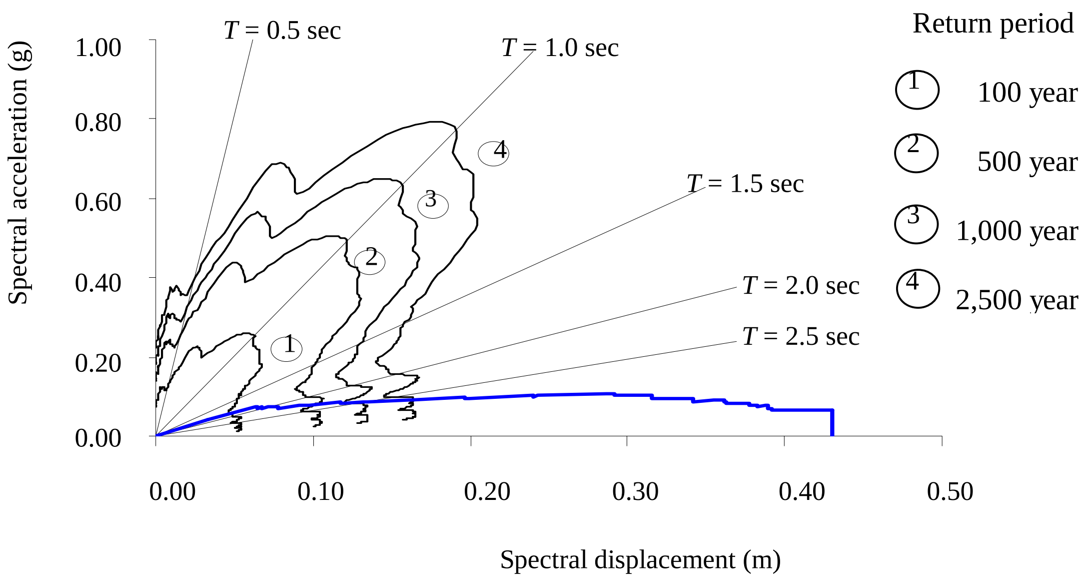


Fig.11. Comparison of capacity and demand spectrum for difference level of intensity of earthquake ground motions for ductility ratio equal 1

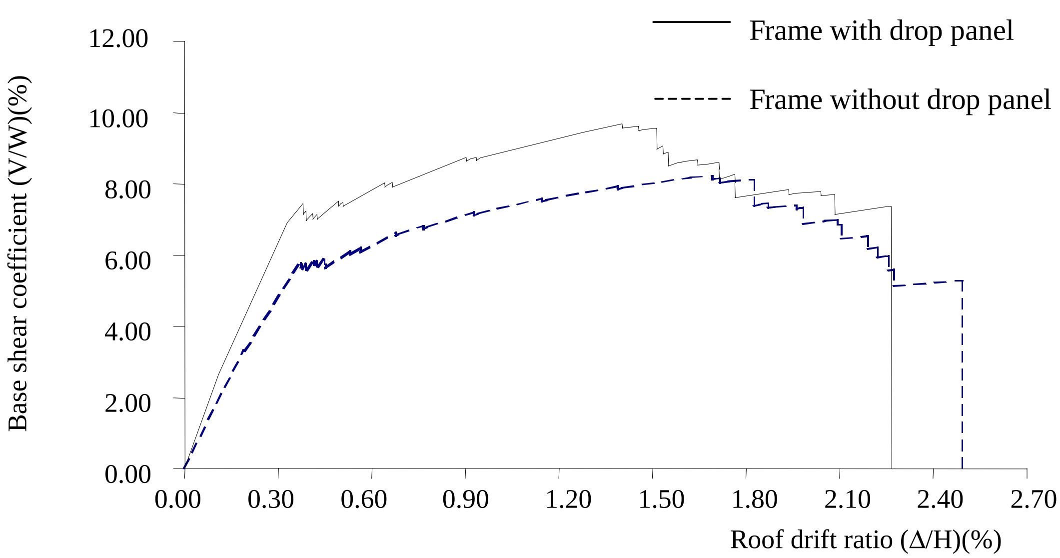


Fig.12. Effect of drop panel on seismic capacity

Shear wall

Shear wall with height 23.4 meters width 2.5 meters and thickness 0.3 meter is introduced to the building. The effect of shear wall on seismic capacity is shown in Fig.13. The result show that the lateral capacity of the building increase about 40 %. It is can be seen that the shear wall can increase the strength and stiffness of the slab-column building significantly.

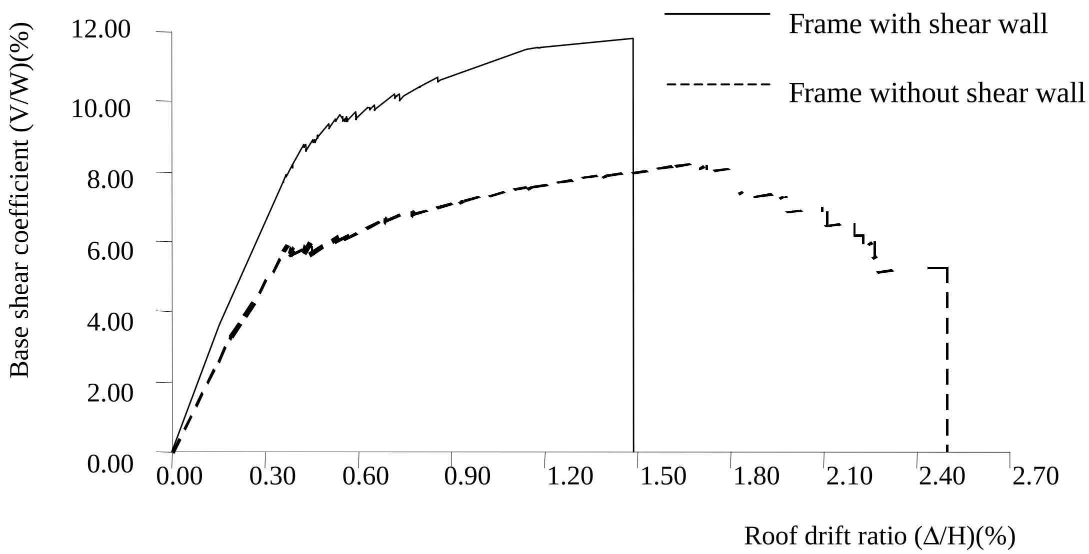


Fig 13. Effect of shear wall on seismic capacity

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 analysis 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. Form numerical examples of the 9-story post-tension flat-plate building in Bangkok, the conclusions are summarized as follows:

1. Sequence of yielding and failure of slab-column frame can be clearly seen by the nonlinear pushover analysis. The flat-plate building clearly behave the strong column-weak beam mechanism. The maximum base shear coefficient is about 8% while the maximum roof drift ratio is 2.5% and the maximum local drift ratio is 3.8%.

2. 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.

3. In this building, the pile foundations are relatively stable and do not significantly affect the building capacity and response.

4. For ground motions of 10% (500 year return period) probability of exceeding in a 50-year exposure period, the building deforms into inelastic range which leads to failure of masonry infill wall, flexural yielding of slab-beams, and yielding of slab-column connections. However, this building will not collapse when it is subject to the highest intensity earthquake ground motions expected in Bangkok despite the fact that the building was designed without any consideration on seismic loading.

5. 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 building can increase about 18% and 40% by applying the drop panel and shear wall, respectively.

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