SEISMIC PERFORMANCE EVALUATION OF REINFORCED-CONCRETE BUILDINGS BY STATIC PUSHOVER AND NONLINEAR DYNAMIC ANALYSES

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ABSTRACT :

The prediction of inelastic seismic responses and the evaluation of seismic performance of a building structure are very important subjects in performance-based seismic design. The seismic performances of reinforced-concrete buildings evaluated by nonlinear static analysis (pushover analysis and modal pushover analysis) and nonlinear time history analysis are compared in this research. A finite element model that can accurately simulate nonlinear behavior of building is formulated by considering several important effects such as p-delta, masonry in-fill walls, soil-structure interaction, and beam-column joints that can be considered rigid zones with joint failure due to poor detailing of joints. Both global response such as system ductility demand and local response such as inter-story drift are investigated in this research. A numerical example is performed on a 9-story reinforced concrete building in Bangkok. Because Bangkok is located in soft to medium soils, response of studied building under a simulated earthquake ground motion at Bangkok site is compared with that under a measured earthquake ground motion of EI-Centro. Finally, the global and local responses obtained from the modal pushover analysis are compared with those obtained from the nonlinear dynamic analysis of MDOF system. The results show that the MPA is accurate enough for practical applications in seismic performance evaluation when compared with the nonlinear dynamic analysis of MDOF system. The results also show that ductility of the studied building can be estimated to 2.40, 2.02 and 1.65 by Fajfar, Chopra and Lee methods, respectively, for simulated ground motion at Bangkok site for a 500-year return period.

KEYWORDS: Seismic Evaluation, Pushover Analysis, Dynamic Analysis, Reinforced-Concrete Buildings

1. INTRODUCTION

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 the Ministerial Law in 2007, many existing reinforced concrete 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.

Presently, there are two methods for investigating inelastic seismic performance. One is the nonlinear time history analysis and another is nonlinear static analysis called "pushover analysis". The nonlinear time history analysis can be divided into two methods. One is based on the dynamic response of an equivalent single degree of freedom system derived from a multi degree of freedom (MDOF) system [Fajfar, 2002]. The other is based on the equivalent response directly obtained from the nonlinear dynamic response of a MDOF system [Lee et al., 2006]. The static pushover analysis can also be divided into two methods. One is based on the first-mode pushover analysis [ATC-40, 1996; FEMA 1997]. The other is based on the modal pushover analysis (MPA) where higher mode effects are taken into account [Seneviratna et.al, 1997; Chopra and Goel, 2002].

In this study, the seismic performances of 9-story reinforced-concrete building are evaluated and compared by the nonlinear static pushover analysis and nonlinear dynamic analysis.

2. MODAL PUSHOVER PROCEDURE

The equation of motion for a symmetric-plan multistory building subjected to earthquake ground motion acceleration $\ddot{u}_g(t)$ are the same as those for external forces, known as the effective earthquake forces [Chopra, 2001].

$$p_{eff}(t) = -mi\ddot{u}_g(t) \tag{1}$$

where is m the mass matrix and i is a vector with all elements equal to unity. Defined by $s = mi$, the spatial (height-wise) distribution of forces can be expanded into its modal components s_n

$$s = \sum_{n=1}^{N} s_n \qquad s_n = \Gamma_n m \phi_n \tag{2}$$

where ϕ_n is the nth mode and $\Gamma_n = \phi_n^T mi / \phi_n^T m \phi_n$

In the MPA procedure, the peak response of the building to $p_{eff,n}(t) = -s_n \ddot{u}_g(t)$, the nth mode component of effective forces, is determined by a nonlinear static or pushover analysis. The peak demands due to these modal components of forces are then combined by an appropriate modal combination rule. The steps are summarized as a series used to estimate the peak inelastic response of higher modes of vibration [Chopra and Goel, 2002].

3. 9-STORY REINFORCED CONCRETE FRAME BUILDING

3.1 Building Details

This building is a 9-story reinforced-concrete structure located in Bangkok, Thailand designed only for gravity loads. The rectangular plan of building is 14.40 meters by 35.10 meters. The story height is 2.50 meters with a total height of 22.50 meters. The structural system is essentially symmetrical. The frames of building were designed as gravity frames. All columns: 0.30 meters by 0.50 meters, 0.30 meters by 0.40 meters, and 0.25 meters by 0.40 meters; are rested on pile cap supported by a group of driven piles with 4 piles. Each pile is of I-shaped 0.30 meters in size and 21 meters in length. It is designed for a vertical safe load of 40 tons. The cylinder compressive strengths of concrete columns and beams are 24 MPa. The expected yield strength of steel deformed and rounded bars are 300 MPa and 240 MPa, respectively.

3.2 Joint Model

A finite element model of building that can accurately simulate nonlinear behavior of building is formulated by considering several important effects such as p-delta, masonry in-fill walls, soil-structure interaction, and beam-column joints (Figure 1) that can be considered rigid zones with joint failure due to poor detailing of joints, shear failure in columns and beams. More details are given by Choopool [2004].

3.3 Foundation Model

Behavior of foundation components and effects of soil-structure interaction were investigated. 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. In this study, the subgrade-reaction model originally proposed by Winkler in 1967 which can be represented by a series of uncoupled lateral and axial springs simulating soil-pile interaction was used in order to model the behavior of foundations. The force-deformation relation of the 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. Details for the Winkler model are given by Boonyapinyo et al. [2006].

Based on the subgrade reaction model and the above assumption, a pile is modeled by three models as shown in Figure 2. Pile model 1 (Figure 2 a) is fully modeled for soil-pile-structure interaction. Moreover, the flexural hinge having moment-rotation relation is introduced in this model, along the pile length, to present the flexural behavior of reinforced concrete pile under lateral load. Some piles in the group were lumped such that 4-pile group was reduced into a 2-pile group. Pile model 2 (Figure 2 b) is considered in 2D frame analysis, a column of piles in the group were lumped into a single section material property using the principle of parallel materials. Therefore, a four-pile group was modeled as a two-pile group. Pile model 3 (Figure 2 c) is considered in 3D frame analysis.

The effect of foundation stiffness on the capacity of the building was also evaluated. The capacity curves for flexible based and fixed based building are plotted in Figure 3 for the 3 different Winkler models. At the same load level, the roof displacement of flexible support is slightly higher than 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 was relatively stiff and did not significant affect the building capacity and response.

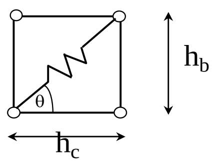
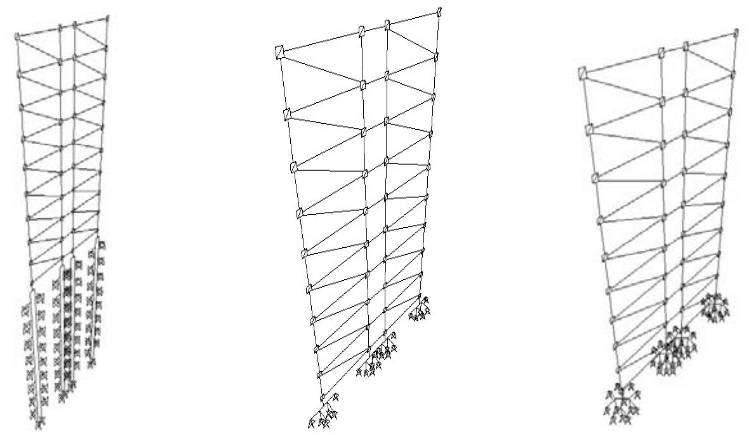


Figure 1 Rigid beam-column joint modeling



(a) Pile model 1 (b) Pile model 2 (c) Pile model 3

Figure 2 Refined pile models

4. COMPARISON OF MPA AND NONLINEAR DYNAMIC PROCEDURE

In this study, nonlinear modal pushover analyses (MPA) were performed by using SAP2000 software [SAP2000].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. 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].

For this building, computed total story stiffness and mass values are given in Table 1. These values were used to calculate the elastic modal attributes of the system including modal periods, modal participant factors and modal mass ratio for the first third modes as shown in Table 1. The corresponding modal shapes and lateral forces are shown in Figure 4 and Figure 5, respectively. It was determined that the modal mass in the first third modes is 76.4, 8.6 and 4.1%, respectively. The first three modes are about 89.10% of the modal contributions to the response.

The modal capacity curves for the 9-story building are generated using an invariant load vectors based on individual mode shapes. The invariant load vector is pattern as $S_n = m\phi$, where m is modal mass matrix and ϕ is the mode shape of the nth mode. Pushover analyses were conducted using the load vectors. The resultant modal capacity curves in terms of normalized base shear with reactive weight versus roof drift ratio are presented in Figure 6. Roof and local drift ratio profiles in each mode obtained at the end of each pushover analysis are present in Figure 7 and Figure 8, respectively. It is noted that resultant deformed shapes are in strong agreement with shape of applied load vectors.

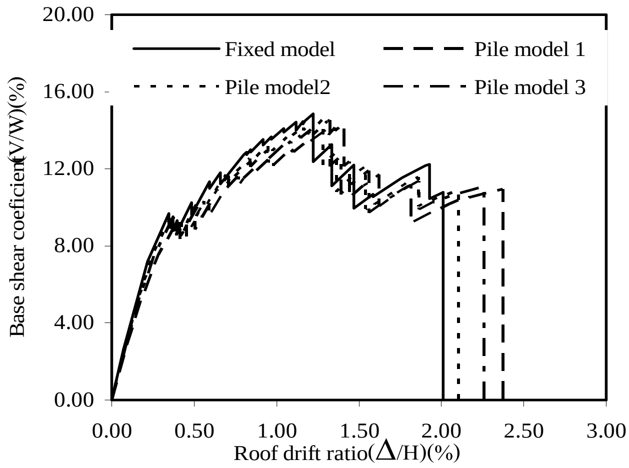
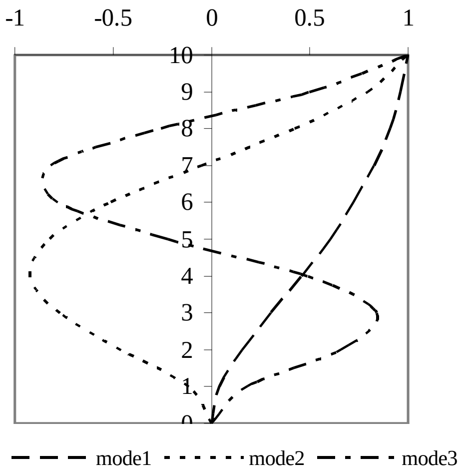


Figure 3 Effect of pile on capacity (Winkler Model)

Table 1 Elastic modal properties of 9-story reinforced-concrete building

9-Story Building	Mode1	Mode2	Mode3
Modal periods, T_n	1.32	0.45	0.33
Modal participant factors, Γ_n	172.69	58.04	40.22
Modal mass(%)	76.4	8.6	4.1



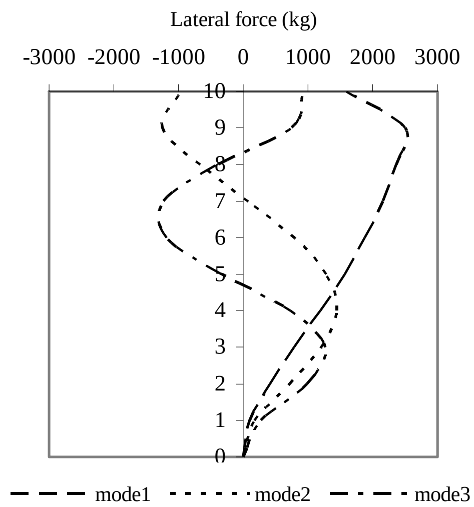


Figure 4 Elastic modal shapes of the first third modes **Figure 5** Lateral forces used in modal pushover

Nonlinear time history analyses were performed on this building with 2% damping using SAP2000 software [SAP2000] called modal response history analysis (MRHA). The building analysis was performed using data from El Centro, California, during the Imperial Valley earthquake of May 18, 1940 and simulated ground motions at Bangkok site proposed by Kiattivisanchai [2001]. The selected ground motions are scaled in such a manner so that the resulting peak roof displacement is equal to the target roof displacement for the building. In this study, the scaled factor for El Centro and simulated ground motions at Bangkok site are 2.0 (PGA=0.63g) and 0.16 (PGA=0.16g), respectively. The target roof displacement is 1.24% of roof drift for the first mode (see Figs.6 &7). Roof and local drift ratio profiles in all modes compared to modal response history analysis are shown in figure 9 and figure 10, respectively.

The displacement amplification at the upper stories is obvious in the higher modes. The first mode alone provides adequate estimates of floor displacements but it is inadequate, especially in estimating the story drift. One mode pushover analysis was unable to identify the plastic hinges in the upper stories where higher contributions to response are known to be more significant. The higher modes are necessary to identify hinges in the upper stories. When compared to nonlinear dynamic analysis, MPA including three modes slightly overestimates the story drift of the lower floors and generally agree well for the story drift of the upper floors for simulated ground motions at Bangkok site.

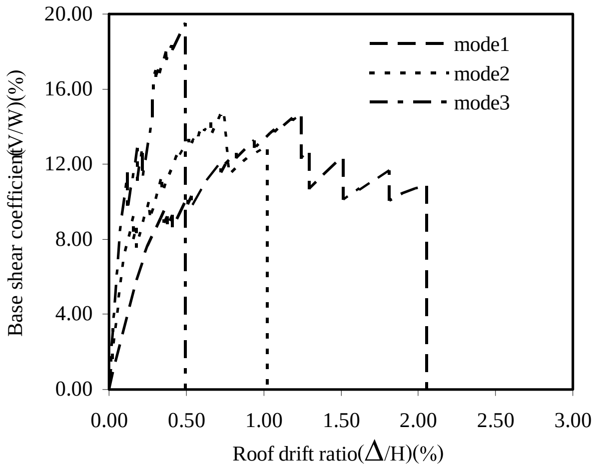


Figure 6 Capacity curve for 9-story building based on separate pushover analysis

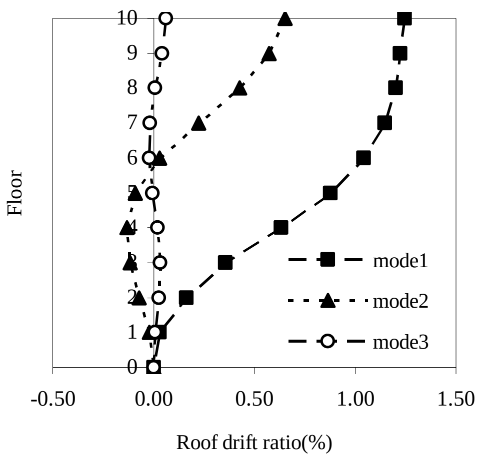


Figure 7 Roof drift profiles in each mode for 9-story building

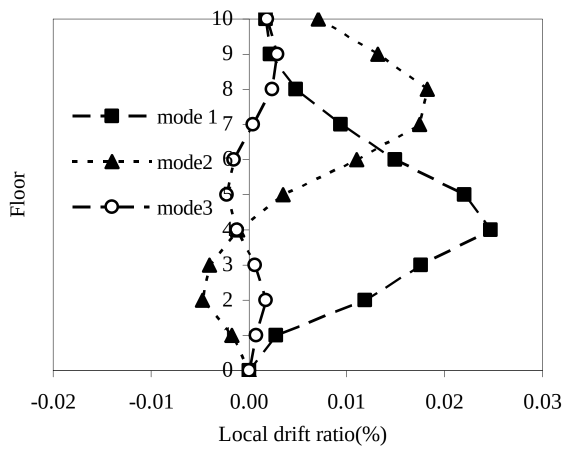
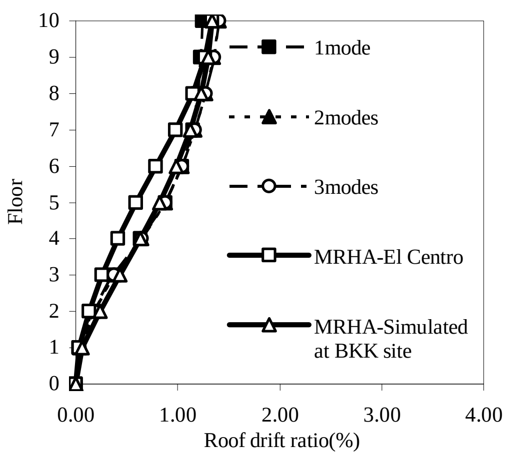


Figure 8 Local drift profiles in each mode for 9-story building



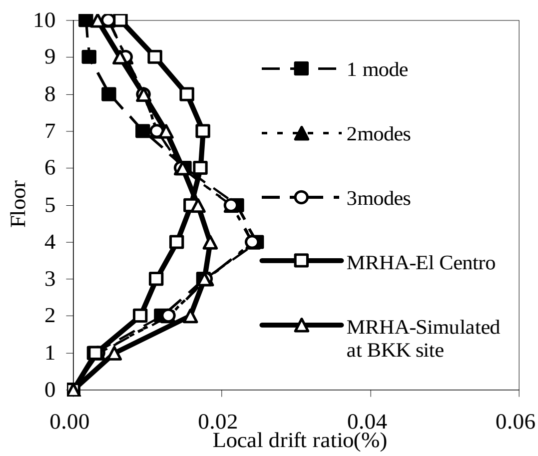


Figure 9 Roof drift profiles for 9-story building **Figure 10** Local drift profiles for 9-story building
(Excitations are 2 x the El Centro and 0.16 x the simulated ground motion at Bangkok site)

5. SEISMIC PERFORMANCE EVALUATION

Since the evaluation of the inelastic design spectrum based on the effective damping is an indirect and ambiguous method, this approach has some problems as an evaluation method of seismic capacity of a structure [Chopra and Goel, 1999]. Recently, several researchers have implemented the concept of constant ductility design spectrum ($R-\mu-T$) for the evaluation of a demand spectrum and have shown that the use of inelastic demand spectra leads to much better estimations of the inelastic displacement demand than those from the elastic demand spectra based on the effective viscous damping in the existing capacity spectrum method.

5.1Estimation of Performance Point Based on Equivalent SDOF Responses

The capacity spectrum is the form of the SDOF system estimated considering the dynamic parameters such as modal participant factor and effective mass for the first mode. The seismic demand for the equivalent SDOF system can be determined by using the graphical procedure. Both the demand spectra and the capacity diagram have been plotted in the same graph. The system ductility demand is estimated by the ratio of yield displacement in capacity spectrum to inelastic maximum displacement in the performance point.

Fajfar (Fajfar, 2000) presented the intersection of the radial line corresponding to the elastic period of the idealized bilinear system, T^*, with the elastic demand spectrum, S_{ae}, defines the acceleration demand required for elastic behavior and the corresponding elastic displacement demand. The yield acceleration, S_{ay}, represents both the acceleration demand and the capacity of inelastic system. The reduction factor, R_μ, can be determined as the ratio between the accelerations corresponding to the elastic and inelastic system.

$$R_\mu = \frac{S_{ae}(T^*)}{S_{ay}} \tag{3}$$

If the elastic period, T^*, is larger than or equal to T_c; the transition period where the constant acceleration segment of the response spectrum passes to the constant velocity segment of the spectrum, the inelastic displacement demand, S_d, is equal to the elastic displacement demand S_{de}. The ductility demand, define as $\mu = S_d / D_Y^*$, is equal to R_μ.

$$S_d = S_{de}(T^*) \qquad T^* \geq T_c \tag{4}$$

$$\mu = R_\mu \tag{5}$$

If the elastic period, T^*, is smaller than T_c, the ductility demand can be calculated as shown in Eq.(6).

$$\mu = (R_\mu - 1)\frac{T_c}{T^*} + 1 \quad T^* < T_c \tag{6}$$

The displacement demand can be determined as

$$S_d = \mu D_y^* \tag{7}$$

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 11. In this study, the peak ground acceleration for Bangkok site for 10% (500-year return period) probability of exceedance in a 50-year exposure period is 0.14g, as suggested by Warnitchai et al. [2000]. The elastic period is larger than T_c. By using Fajfar method, the equal displacement rule applies system ductility of this building to be 2.40. The displacement demand of the equivalent SDOF system is transformed back to the top displacement of the MDOF system equal to 14.0 cm. The building deforms into the inelastic range which lead to yielding of some beams.

Chopra [Chopra and Goel, 1999] presented the performance procedure by graphical method with iteration. The static equivalent displacement and the resistant force obtained from the pushover analysis of the method using Lee et al. equations [2006] as shown in Eq.(8).

$$S_d = D_{equi} = \frac{\phi_1^T M X_{static}}{\phi_1^T Mi} \qquad S_a = \frac{V_{equi}}{M_{eff}} = \frac{\phi_1^T R_{static}}{M_{eff}} \tag{8}$$

where X_{static} and R_{static} are displacement and resistant force for each story from pushover analysis respectively.

M is diagonal mass matrix, M_{eff} is the effectively mass of the first mode of the building, S_d and S_a are equivalent spectral displacement and spectral acceleration estimated by the inelastic static response.

The performance point of capacity and demand spectrum in ADRS format for different levels of intensity of earthquake ground motions in Bangkok are shown in Figure 12. The equal displacement rule applies system ductility of this building to be 2.02. The displacement demand of the equivalent system is equal to 9.10 cm. The building deforms into the inelastic range which lead to yielding of some beams.

5.2 Estimation of Performance Point Based on Equivalent MDOF Responses

Lee [Lee et al., 2006] proposed the performance procedure in the inelastic time history analysis of MDOF system to estimate the global system ductility demand and the local inelastic seismic response. This method is directly evaluated by the relationship between the equivalent displacement and force of a multistory building without converting the MDOF system into ESDOF system in Eq.(9) and Eq.(10).

$$D_{equi} = \frac{\phi_1^T MX(t)}{\phi_1^T M1} \tag{9}$$

$$V_{equi} = \phi_1^T R(x) = \phi_1^T KX\ (t) = \phi_1^T K\phi_1 D_{equi} = k_1 D_{equi} \tag{10}$$

where $X(t)$ is the displacement response obtained from the inelastic time history analyses , $R(x)$ is the resistant force obtained from the inelastic time history analyses and k_1 is stiffness for the 1-st natural mode.

The system ductility demand of earthquake ground motions in Bangkok is equal to 1.65 as shown in Figure 13. The displacement demand of the system is equal to 9.00 cm.

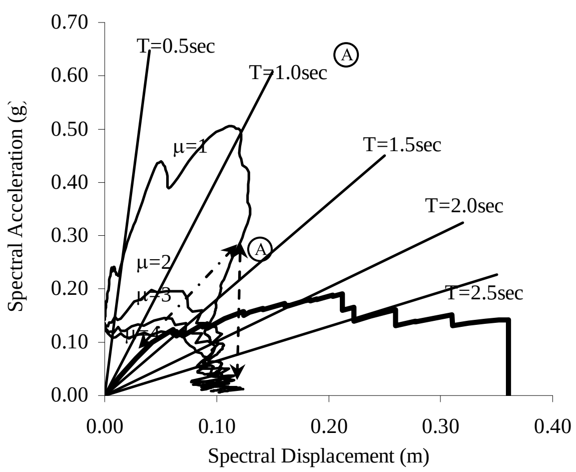


Figure 11 Comparison of capacity and demand spectrum for simulated ground motion at Bangkok for a 500-year return period for ductility ratio equal to 1-4 (Fajfar method)

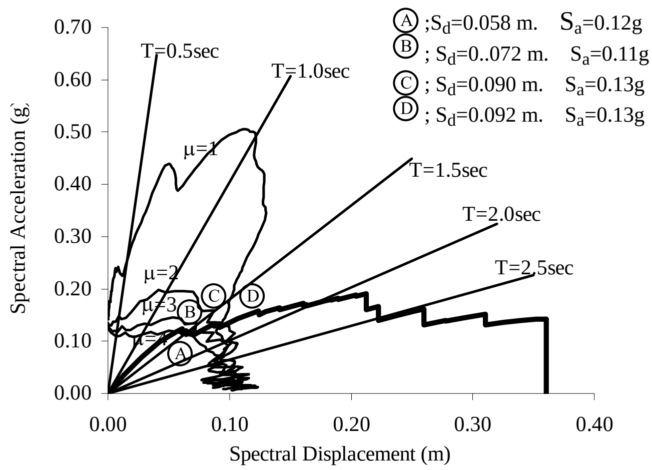


Figure 12 Comparison of capacity and demand spectrum for simulated ground motion at Bangkok for a 500-year return period for ductility ratio equal to 1-4 (Chopra method)

6. CONCLUSIONS

From the numerical example of 9-story reinforced-concrete building, the following conclusion can be drawn.

1. The 9-story reinforced concrete building deforms into the inelastic range which leads to yielding of some beams at Bangkok site for 500-year return period. However, the building will not collapse when subjected to this earthquake ground motions expected in Bangkok despite the fact that the building was designed without any consideration for seismic loading. The ductility of the building can be estimated to be 2.40, 2.02 and 1.65 by Fajfar, Chopra and Lee methods, respectively.

2. The pile foundations were relatively stiff and did not significant affect the building capacity and response.
3. The selection of an appropriate load shape for any nonlinear static procedure is the key issue in accurate prediction of the structural responses.
4. Seismic demands of high-rise buildings can be remarkably improved by considering higher modes.
5. When compared to nonlinear dynamic analysis, MPA including three modes slightly overestimates the story drift of the lower floors and generally agree well for the story drift of the upper floors for simulated ground motions at Bangkok site.

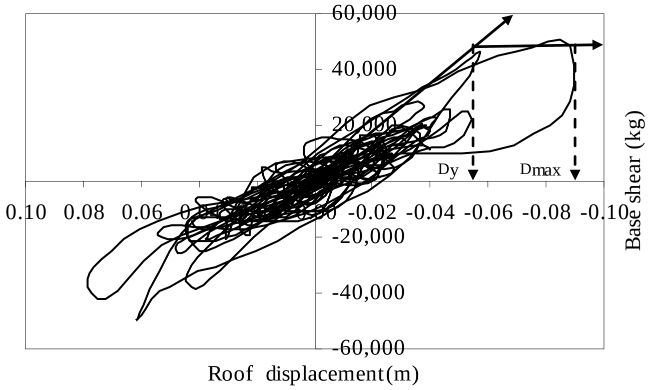


Figure 13 System ductility demand for simulated ground motion at Bangkok for a 500-year return period (Lee method**)**

7. ACKNOWLEDGEMENT

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