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Seismic Behavior and Dissipated Plastic Energy of Performance-Based-Designed High-Rise Concrete Structures with Considering Soil–Structure Interaction Effect

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ABSTRACT: Since the structure and foundation are built on soil, the soil is the major platform by which seismic vibrations are transmitted to the structure, and has noticeable effects on the response and behavior of structure during earthquakes. In this research, the recently introduced Performance-Based Plastic Design (PBPD) and its Modified Performance-Based Plastic Design (MPBPD) method in which soil and structure interaction effect has been considered underwent the seismic evaluation. In order to do evaluation, a twenty-floor concrete structure with MPBPD method and conventional PBPD was designed and analyzed in accordance with the time history of the 22 far-field quake records. In this study, cone model is employed for modeling the soil and foundation. With a detailed three-dimensional finite element model of a twenty-story high-rise structure constructed and exploited in the OpenSees software, it is attempted to consider a more realistic behavior of the structure. The results of six related parameters with the maximum response of the structure demonstrate the efficiency and performance of the MPBPD method for the purpose of considering the SSI effect, compared with the conventional method of PBPD. The Results show that, in the MPBPD design method, maximum displacement, acceleration, inter-story drift and shear force dropped leading to a better distribution of energy in the structure compared to the PBPD method.

Keywords: Performance-Based Plastic Design (PBPD), Reinforcement Concrete Structure, Seismic Energy Dissipated, Soil-Structure Interaction (SSI), Special Moment Frames.

INTRODUCTION

In conventional methods of design structures under seismic loads, the inelastic behavior of structures, by using seismic behavior factor and magnification factor displacement, is indirectly considered in design process. The outcome of this design approach is a seriously uncontrolled yielding in members, buckling structural elements, ruptures and local instability, that can occur largely and non-uniformly in structure eventually resulting in an unpredictable and undesirable responses so that, with the death toll which occurred from the last earthquakes in buildings designed with available structural codes, a

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kind of awareness is created in the society of engineers which says that the existing seismic design methods cannot provide the desired safety and efficiency. Thus, there is a need for a design approach such as performance-based plastic design that leads to desired performance and predictive structural response which is strongly felt (Sahoo and Rai, 2013). In this regard, several methods of design such as, capacity spectrum method presented by Freeman (2004), N-2 method introduced by Fajfar (2000), yield point spectra method presented by Aschheim and Black (2000), modified lateral force procedure introduced by Englekirk (2003) and developed by Panagiotou and Restrepo (2008), Direct Displacement-Based Design (DDBD) developed by Priestley et al. (2007) and Panagiotou (2008), and PBPD method (Bai et al. 2018) have been developed. The main idea of PBPD is the consideration of this issue that, the actual response of structure is associated with the design philosophy and construction of structure and not the type of mathematical analysis.

Unlike the present methods in design codes, in PBPD, the design base shear selected for a risk level is obtained by equating the work necessary to bring the structure to the target drift uniformly with the energy needed in a structure with one degree of freedom. The structure designed in this manner will act based on the performance limit states such as target drift and desired yield mechanism. The nonlinear behavior of structures is directly considered, and it practically eliminates the evaluation and repetition necessity by a nonlinear static analysis or time history analysis after the initial design. More comprehensive discussions as well as the theoretical justification can be found in Goel and Chao (2008). However, PBPD design approach is provided for modeling the structure as the fixed base while SSI effect can significantly affect the response of the structure.

During the studies conducted over the past three decades on the dynamic characteristics and seismic response of structures, the importance of SSI is well specified particularly for heavy and hard structures that are constructed on soft and relatively soft soils. The existing conventional structural codes cannot guarantee the safety of the buildings constructed on soft and relatively soft soils (Panagiotou, 2008; ASCE, 2010). Previous events indicate several severe structural damages which occurred due to neglecting the effect of SSI during the earthquake. SSI often results in a change in the dynamic and seismic specifications of structures (Lou et al., 2011); it can also change the response of the structure, and lead to corruption and loss of performance criteria that are expected from the structure. Among these specifications, we stated the increase in structural vibration period and structural damping change. The studies that have been performed to evaluate the effect of SSI on performance-based design methods are very limited, and most of them have examined the performance-based design methods in a general form. Fatahi et al. (2011) have studied the effects of SSI on a 15-story two-dimensional structure with a moment frame system and perfectly elasto-plastic behavior under four earthquakes. Through the investigation of three types of soil with shear wave velocity less than 600 m/s and the structure drift and displacement response, they have concluded that the structures conventional non-linear design process cannot guarantee the safety of structures constructed on soft soils. They also suggested that SSI effect must be considered in the design of structures constructed on soft soil. Other studies that examined the effect of SSI and performance of the structures, have confirmed the necessity of modification and considered the SSI effects in the design process (Tabatabaiefar et al., 2012; 2015). In a recent study by Rezaie and Mortezaie

(2017), maintaining the simplicity of the conventional PBPD, by modifying two important parameters, the vibration period of the structure and the structure target drift, the design base shear force due to the interaction of the soil and structure in PBPD method has been modified and the design process is presented for this purpose. Here, the modified method provided by Rezaie and Mortezaie (2017) is briefly referred to as MPBPD. (Rezaie and Mortezaie, 2017; Mortezaie and Rezaie, 2018).

Investigating the history of research, the authors of this paper, have found that so far, seismic analysis has not been carried out considering the effects of soil and structure effects on either the PBPD design or the MPBPD method. Therefore, in order to evaluate the behavior of the designed structure by PBPD and MPBPD methods, in addition to the two parameters of displacement and drift of structures that have been investigated in previous studies to evaluate the structural efficiency, the plastic energy lost, structure base shear, the number of plastic joints formed, and relative acceleration of the floors have been studied. Time history analysis was carried out by constructing a precise and complex three-dimensional model of the twenty–floor concrete under a series of far-field earthquake records, with the inclusion of a cone-shaped soil model.

DESIGN PROCEDURE

Summary of the Current PBPD

This design method can be summarized in five steps as follows (Bai et al., 2017; Rezaie and Mortezaie, 2017):

Step 1: Proportion to the desired performance objectives at the risk level of the design earthquake, select a desired yield mechanism and a target drift. (The behavior of the structure is assumed to be elasto-plastic at this step)

Step 2: The period of the structural vibration is estimated, and an equation is chosen for the appropriate distribution of earthquake load into the structure height.

Step 3: The design base shear for a selected risk level is achieved by equating the work necessary to bring the structure to the target drift uniformly, with the energy required in the equal single degree of freedom structure, with elasto-plastic behavior. (Figure 1)

Step 4: If the behavior of structural materials is not in line with elasto-plastic behavior, the design base shear should be corrected at this step.

Step 5: Designing members which are entering the nonlinear behavior in the selected mechanism (such as beams in reinforced concrete moment frame) will be performed using a plastic design method and the members which remain elastic (such as columns) will be designed by the capacity method.

Determination of Design Base Shear

Design base shear in the PBPD method for a given level of risk is achieved based on the inelastic state of the structure and by controlling the drift. Therefore, it is not necessary to separate drift control. The concept of energy employed in PBPD method is similar to the approach used by Housner about 60 years ago (Abdollahzadeh and Mirzagoltabar, 2017; Liao et al., 2017). He applied the difference between the input energy and elastic strain energy to obtain the plastic energy used as the amount of energy absorbed by the structure to design the yielding members. For the seismic design of structures, Housner demonstrated that the pseudo velocity response spectrum remains constant for the common earthquakes in a wide range of periods. He suggested the following formula (Eq. (1)) to estimate the earthquake input maximum total energy for a system having one degree of freedom:

$$E_{total} = E_{plastic} + E_{Elastic} = \frac{1}{2} M.S_v^2 \quad (1)$$

where, M: is the structure total seismic mass, and S_v: is the spectral velocity obtained from the elastic response spectrum. The subsequent researchers sought to modify the equation provided by Housner, and concluded that the input energy is equal to a coefficient of the Housner equation. This coefficient is known as the energy correction factor represented by γ (Figure 2).

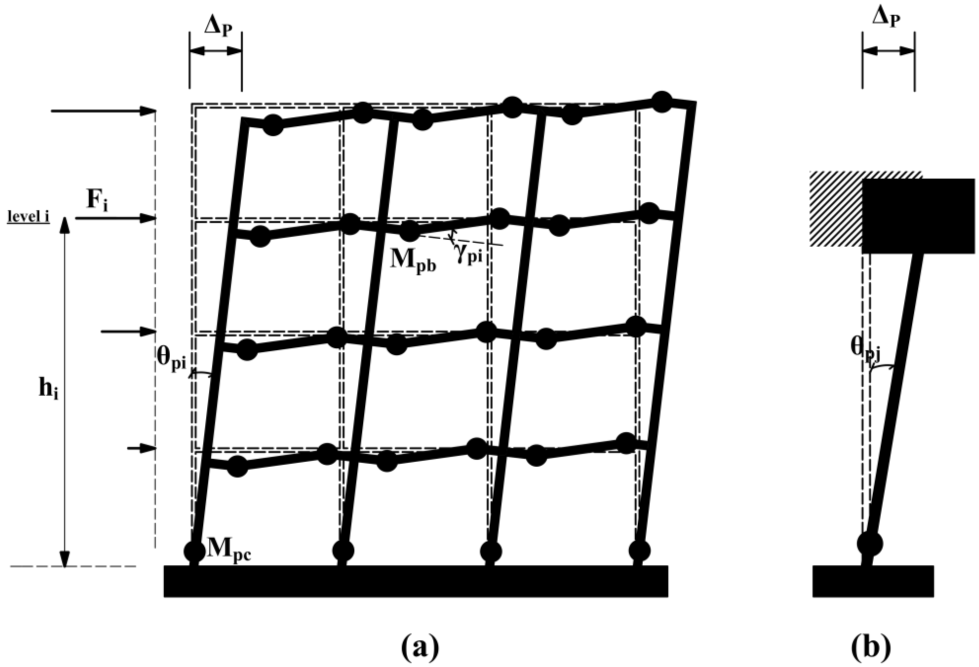


Fig. 1. a) Desirable yield mechanism of reinforcement concrete special moment frame (RC SMF), b) equivalent single degree of freedom structure

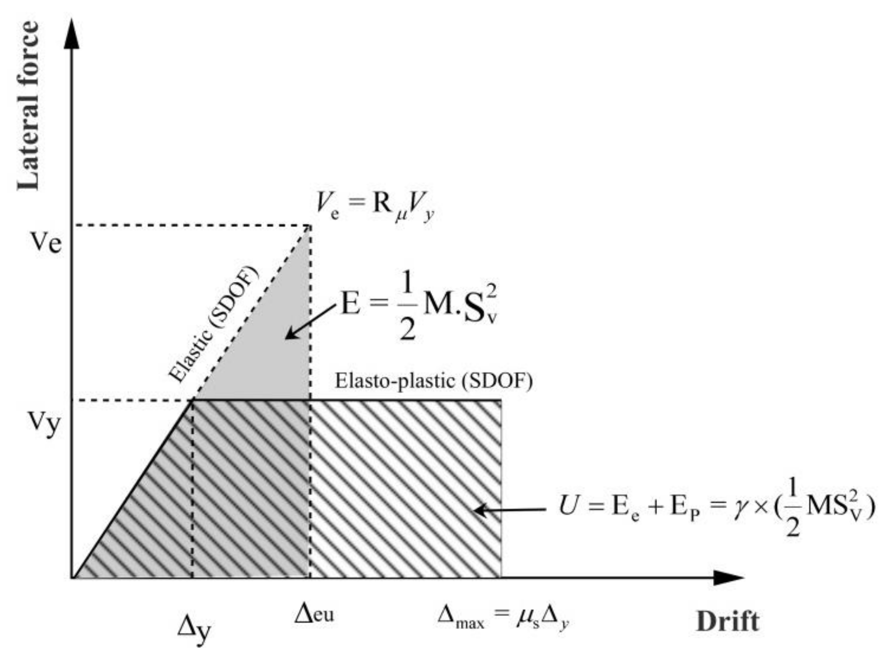


Fig. 2. Concept of PBPD method

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \quad (2)$$

As known, the energy correction factor is dependent on the structural ductility μ_s and ductility reduction factor R_μ , both of which are taken by applying Eqs. (3-4):

$$\mu_s = \frac{\Delta_{max}}{\Delta_y} \quad (3)$$

$$R_\mu = \frac{V_e}{V_y} = \frac{\Delta_{eu}}{\Delta_y} \quad (4)$$

Consequently, Eq. (1) is corrected as follows:

$$E_{total} = \frac{1}{2}\gamma M.S_v^2 \quad (5)$$

in later years, Akiyama and Kato indicated that, assuming the entire structure in the form of a one degree of freedom, the structure elastic energy can be calculated by the following equation with an acceptable accuracy (Alavi et al., 2017):

$$E_e = \frac{1}{2}M\left(\frac{T}{2\pi}.\frac{V_y}{W}g\right)^2 \quad (6)$$

where, V_y : is yield base shear force, T : is main vibration period of structure, g : is acceleration of gravity and W : is structure seismic weight. On this basis, in accordance with Eqs. (5) and (6), structure plastic energy can be achieved by subtracting these two equations:

$$E_p = \frac{WT^2 g}{8\pi^2}\left(\gamma S_a^2 - \left(\frac{V_y}{W}\right)^2\right) \quad (7)$$

By substituting Eqs. (6) and (7) in Eq. (5), we have:

$$\frac{WT^2 g}{8\pi^2}\left(\gamma S_a^2 - \left(\frac{V_y}{W}\right)^2\right) + \frac{1}{2}M\left(\frac{T}{2\pi}.\frac{V_y}{W}g\right)^2 = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}.S_a.g\right)^2 \quad (8)$$

By solving the Eq. (8) for V_y/W , we obtain the following formula for the base shear force in PBPD method:

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}}{2} \quad (9)$$

Such that α is the dimensionless parameter which is equal to:

$$\alpha = \left(\sum_{i=1}^{n}(\varphi_i - \varphi_{i+1})h_i\right).\left(\frac{w_n h_n}{\sum_{i=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}}.\left(\frac{\theta_p 8\pi^2}{T^2 g}\right) \quad (10)$$

in this relation, θ_p : is design plastic drift ratio, φ_i : is share distribution coefficient of the ith story, w_j : is seismic weight of the jth story, w_n : is roof story weight, h_n : is roof height from the foundation, h_j : is height of the j[th] story from the foundation. Other parameters are described in previous equations.

MPBPD Method (Rezaie And Mortezaie, 2017)

The main goals in the PBPD method reform is to keep it simple and apply the existing equations in order to consider the SSI effect. For this purpose, by employing the formula provided in ASCE7-10 (Eq. (11)), the structure vibration period has been modified with regard to the SSI effect. Also, to estimate the lateral drift added due to SSI effect, Eq. (14) presented by Poulos and Davis (1980) is applied to estimate the foundation rotation as a result of soil failure.

Period of Vibration

Considering the SSI effect, vibration period of the structure ($\tilde{T}$) can be achieved using the relation presented in ASCE7-10. It is worth mentioning that in the conventional method of PBPD, the vibration period is obtained from the same codes but for the structure clamped end conditions.

$$\tilde{T} = T\sqrt{1+25\alpha\frac{r_a\bar{h}}{v_s^2T^2}\left(1+1.12\frac{r_a\bar{h}^2}{\alpha_\theta r_m^3}\right)} \qquad (11)$$

in which, r_a and r_m : are foundation-related parameters determined by the following relations:

$$r_a = \sqrt{\frac{A_o}{\pi}} \quad \& \quad r_m = \sqrt[4]{\frac{4I_o}{\pi}} \qquad (12)$$

Also, T : is main vibration period of structure, $\bar{h}$: is effective height of structure, α_θ : is foundation hardness correction factor, V_s : is shear wave velocity, A_o : is foundation bearing surface, I_o : is foundation static moment of inertia around the central horizontal axis and α : is soil-structure density ratio, determined by the following equation:

$$\alpha = \frac{W}{\gamma A_o \bar{h}} \qquad (13)$$

where, γ : is the mean unit weight of soil and W : is seismic weight of structure.

Rotation of Foundation Angle

To estimate the additional drift of structure caused by the soil under the foundation, the following simple equation was applied:

$$\tan\theta_i = \frac{1-\nu^2}{E}\cdot\frac{M}{B^2.L}.I_\theta \qquad (14)$$

where θ_i : is rotation of foundation angle; M : is bending moment; ν and E : are elastic parameters of soil, I_θ : is impact factor of foundation.

According to the above, the modified target displacement angle (θ_u') is:

$$\theta_u' = \theta_u + \theta_i \qquad (15)$$

The added drift and modified vibration period, owing to the effect of SSI, lead to a change in the energy correction factor which itself relies on the structural ductility factor (μ_s) and ductility reduction factor (R_μ). The prime symbols added in the following parameters reveal the modified values of the above parameters as a result of the changes. Hence, energy correction factor is:

$$\gamma' = \frac{2\mu_s' \text{-}1}{(R_\mu')^2} \qquad (16)$$

With these reforms and with regard to Eqs. (6-7) and (9-10), the design base shear is obtained in MPBPD method. The rest of the design steps is similar to the PBPD. In Figure 3, it is attempted to present a pictorial concept of MPBPD method.

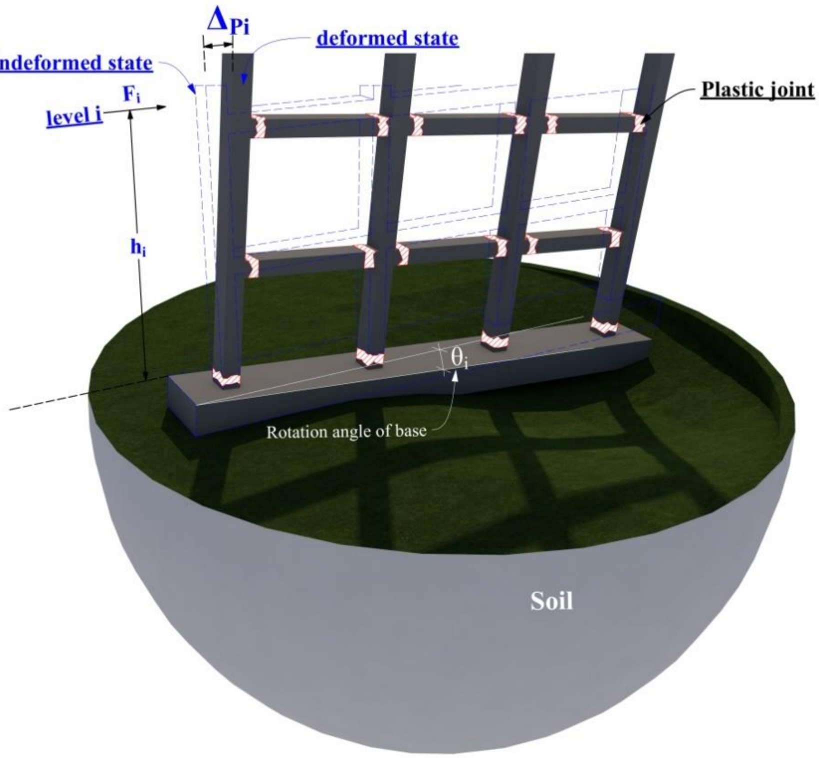


Fig. 3. SSI effect in MPBPD method

DESIGN STRUCTURES

Apart from design methodology, other design assumptions and parameters for redesign work were fixed as the same as those of the frames in FEMA P695 (FEMA, 2009), for stability and fair comparison. Two twenty-story space frames compatible with the conditions and requirements of MPBPD and PBPD methods were designed. Important design parameters have been given in Table 1 and Table 2. Three-dimensional floor plan has been illustrated in Figure 4. The element sizes of two twenty-story structures with characteristics such as reinforcement ratio of beams and column are given in Figure 5.

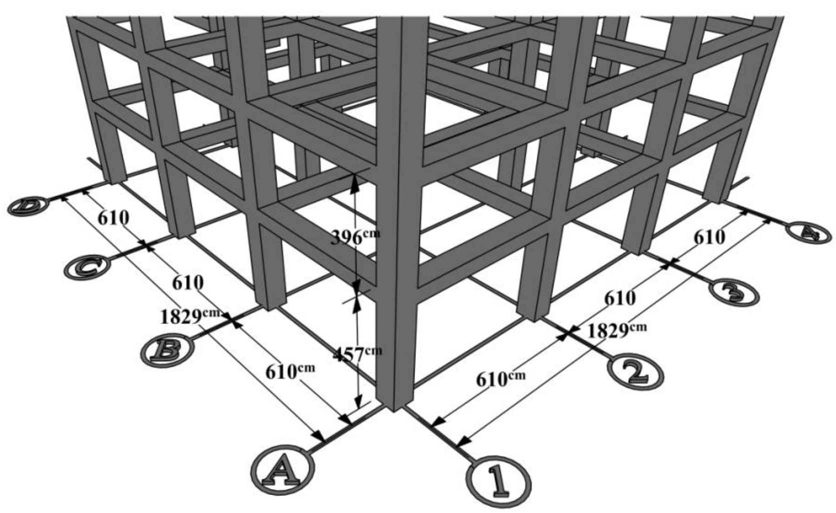


Fig. 4. Specification floor plan and elevation of structure

Table 1. Summary of design parameters for RC SMF FEMA P695 (2009) and specification of soil and foundation (ASCE, 2010)

Parameter	Range Considered
Seismic design level	Design category D
First story and typical upper story heights respectively,	457-356 centimeter
Bay width	610^{cm}
Compressive strength concrete for column and beam respectively,	41.4-34.5 Mpa
Design floor dead load	854.4 kg/m^2
Design floor live load	244.1 kg/m^2
Yield drift Ratio	0.5%
Target drift Ratio	2%
Concrete cracking effect in beams	$0.5EI_g$ (Poulos and Davis 1980)
Concrete cracking effect in columns	$0.7EI_g$ (ASCE 2010)

Table 2. Specification of soil and foundation (FEMA, 2009)

Parameter	Range Considered
Shallow foundation dimensions (length-width-height)	$1829\times1829\times125^{cm}$
Compressive strength concrete for foundation	41.4 Mpa
Average shear wave velocity	182 to 365 m/s
building site (Los Angeles, California)	high seismic site
soil class	S_d
S_m	1.5g
S_{m1}	0.9g
Poisson's ratio ν	0.3
Shear wave velocity	300 m/s

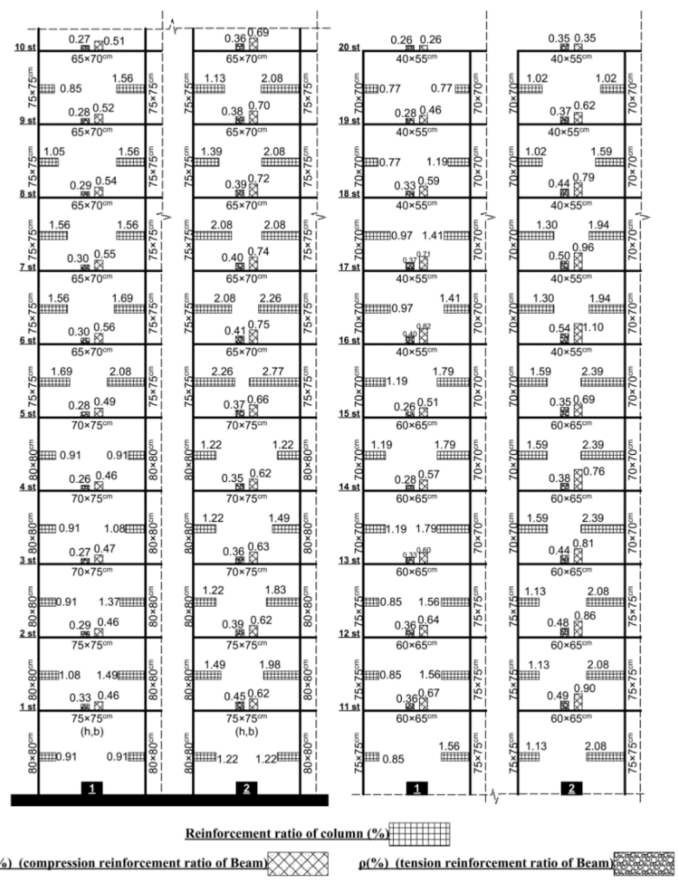


Fig. 5. Detail of design twenty-story space frame in: 1) MPBPD method, 2) PBPD method

STRUCTURAL AND SOIL MODELS

Nonlinear Dynamic Time History Analyses and Ground Motions

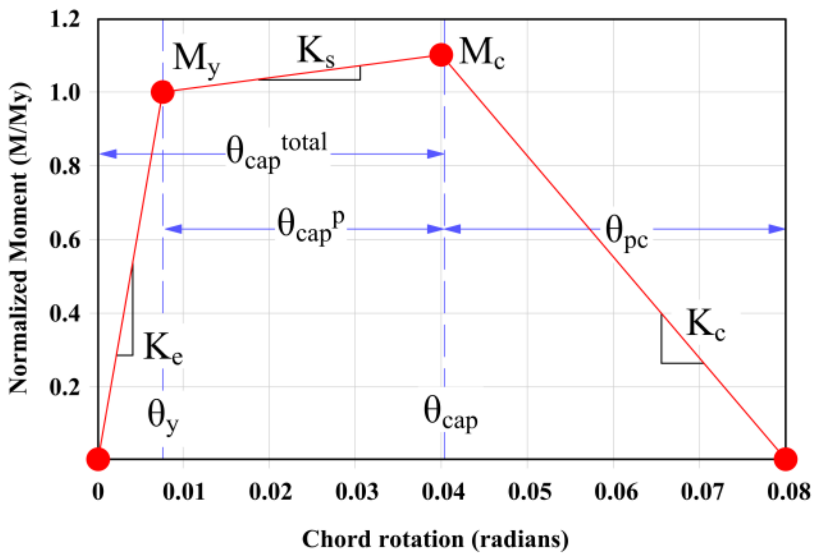
Since the accurate choice of earthquake records is necessary in order to perform a reliable analysis, in this study, nonlinear dynamic time history analyses were conducted under 22 far-field earthquake acceleration records, as introduced in FEMA P695. Each record of earthquake is formed by two components along with x and y. Specifications and features of some of these records are listed in Table 3. Structure of nonlinear modeling based on incorrect and unrealistic methods can lead to incorrect and illogical responses. There are numerous finite element models for concrete structures. However, most of them are not able to simulate the structural failure.

In 2005, hysteresis model for behavior of the reinforced concrete beam-column elements known as Ibarra model, was presented by Ibarra et al. (Figure 6a). This model can consider the important modes of resistance deterioration that can lead to lateral structural collapse. Using the results of 255 laboratory samples of reinforced concrete columns, Haselton et al. (2007) and Lin et al. (2013) calibrated the Ibarra model and gave a full range of equations for the parameters of this model including: initial stiffness (K_e), stiffness after yield (K_s), plastic rotation capacity ($\theta_{cap}{}^{p}$) and post-maximum-resistance rotation capacity (θ_{pc}). In this study, the hysteresis Ibarra model is employed for modeling nonlinear behavior of beam-column elements.

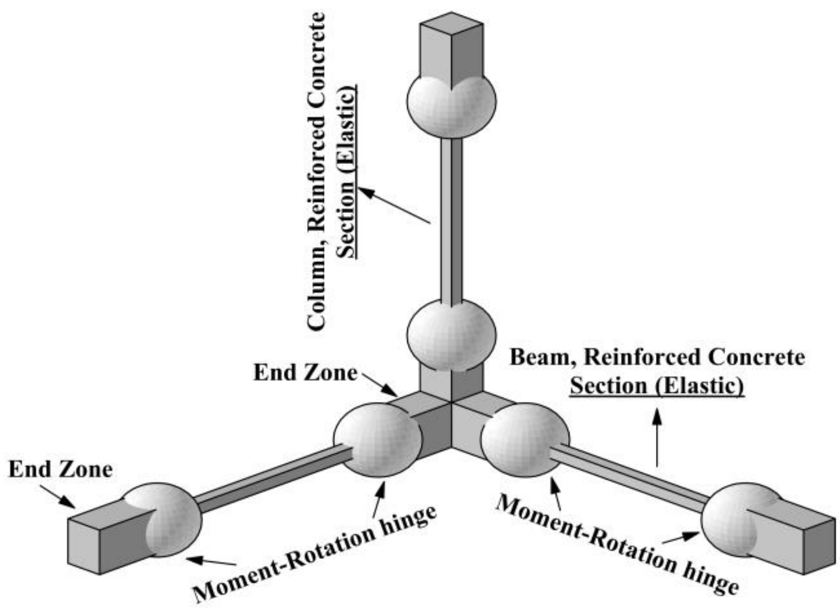
In order to simulate the behavior of structural beam-column, in OpenSees software, the beam-column element formed by an elastic element and two moment plastic hinges focused at two ends and with a length of zero are used (Figure 6b).

Table 3. Far-field ground motion record set used in this study (FEMA 2009)

ID	Name	PGA_{max} (g)	PGV_{max} (cm/s)
1	Cape Mendocino (1992)	0.55	44
2	San Fernando (1971)	0.21	19
3	Friuli Italy (1976)	0.35	31
4	Imperial Valley Delta (1979)	0.35	33
5	Imperial Valley El Centro (1979)	0.38	42
6	Superstition Hills El Centro (1987)	0.36	46
7	Superstition Hills Poe Road (1987)	0.45	36
8	Loma Prieta Capitola (1989)	0.53	35
9	Loma Prieta Gilroy (1989)	0.56	45
10	Landers Coolwater (1992)	0.42	42
11	Landers Yermo Fire Station (1992)	0.24	52
12	Northridge Beverly Hills (1994)	0.52	63
13	Northridge Canyon Country (1994)	0.48	45
14	Kobe Nishi Akashi (1995)	0.51	37
15	Kobe Shin Osaka (1995)	0.24	38
16	Kocaeli Arcelik (1999)	0.22	40
17	Kocaeli Duzce (1999)	0.36	59
18	Chi Chi CHY101 (1999)	0.44	115
19	Chi Chi TCU045 (1999)	0.51	39
20	Duzce Bolu (1999)	0.82	62
21	Manjil Abbar (1990)	0.51	54
22	Hector Mine (1999)	0.34	42



(a)

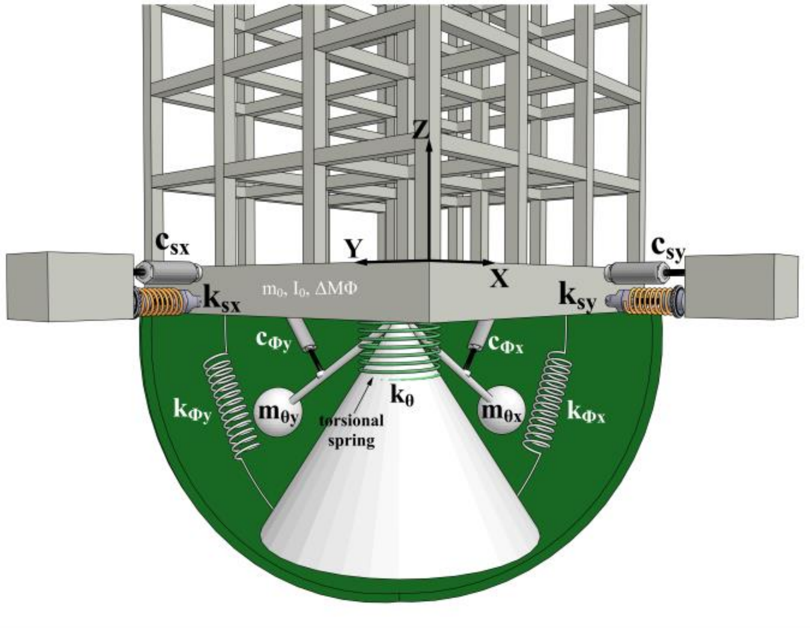


(b)

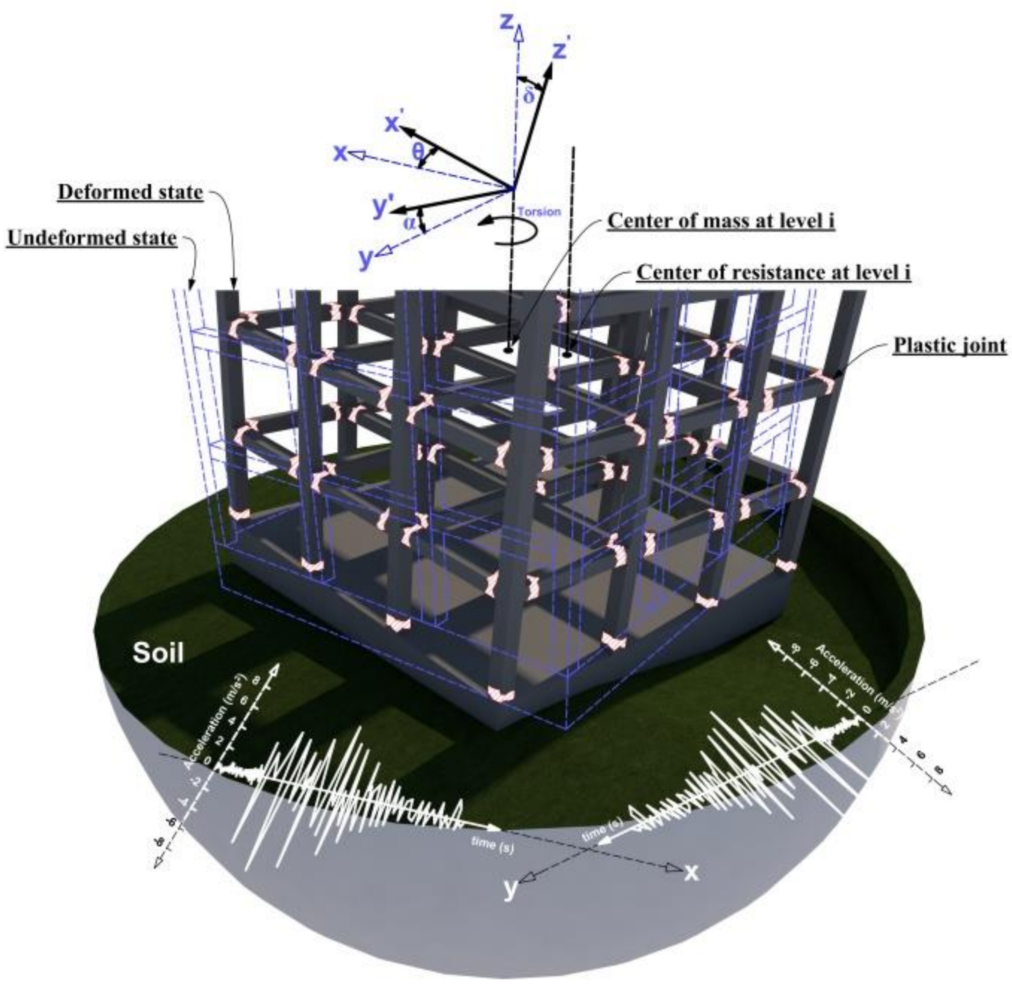
Fig. 6. a) Hysteretic behavior of component model used in this study, b) 3D models of beam-column elements with plastic hinge concentrated at the two element's ends with end rigid zone

Cone Model

In this study, the simplified physical cone model is used for considering the SSI effect (Figure 7a). This model has been formed of a spring, damper and mass that is placed centralized under the foundation. In this model, it is assumed that different degrees of freedom do not interfere with each other. This model has good accuracy in the range of engineering errors and significantly reduces modeling and particularly the time of analysis compared to models based on finite element. According to equations presented by Wolf and Deeks (2004), numeric values of three-dimensional soil cone model are obtained which are equal to the values given in Table 4. Figure 7b illustrates the three-dimensional twenty-story model with effects such as torsion which was considered in the time history analyses.



(a)



(b)

Fig. 7. a) Three-dimensional cone model, b) 3D model of structure under earthquake excitations

Table 4. Key expressions used in model three-dimensional Soil-foundation on the Surface of a homogeneous soil half space (Ibarra et al., 2005)

Motion	Horizontal	Rocking	Torsional
Equivalent radius $r_0(m)$	$\sqrt{\frac{A_0}{\pi}}=10.31$	$\sqrt[4]{\frac{4I_0}{\pi}}=10.43$	$\sqrt[4]{\frac{2I_0}{\pi}}=8.77$
Aspect ratio $\frac{z_0}{r_0}$	$\frac{\pi}{8}(2-\upsilon)=0.66$	$\frac{9\pi}{32}(1-\upsilon)\left(\frac{c}{c_s}\right)^2=1.08$	$\frac{9\pi}{32}=0.88$
Poisson's ratio ν	$all\,\upsilon$	$\upsilon\le\frac{1}{3}$	$\upsilon\le\frac{1}{3}$
Wave velocity c	$c_s=300\,{}^{m}\!/_{s}$	$c_p=396.86\,{}^{m}\!/_{s}$	$c_s=300\,{}^{m}\!/_{s}$
Trapped mass ΔM	0	0	0
Lumped-parameter model	$K_{x,y}=\rho c_s^2\frac{A_0}{z_0}=$ $8.12\times10^{8}\,{}^{kgf}\!/_{m}$	$K_v=3\rho c^2\frac{I_0}{z_0}=$ $7.17\times10^{10}\,{}^{kgf}\!/_{m}$	$K_z=3\rho c_s^2\frac{I_0}{z_0}=$ $5.98\times10^{10}\,{}^{kgf}\!/_{m}$
	$C_{x,y}=\rho c_s A_0=$ $1.84\times10^{7}\,{}^{kgf}\!/_{m}$	$C_v=\rho c I_0=$ $6.79\times10^{8}\,{}^{kgf}\!/_{m}$	$C=\rho c_s I_0=$ $5.13\times10^{8}\,{}^{kgf}\!/_{m}$

ANALYSIS RESULTS

The results of the numerical analyses for the twenty-story structures subjected to 22 earthquake acceleration records are achieved. Given the large volume of information obtained from the analyses and in order to demonstrate the dispersion of results, the mean and standard deviation (SD) for obtained results in two cases: PBPD and MPBPD structures have been calculated and presented in Figures 8 and 9.

SD predicts the variation and dispersion of the data possesses compared to the mean. By examining the numerical results for the two cases (PBPD and MPBPD), it is observed that the maximum of displacement, acceleration, inter-story drift and shear force dropped, and conversely the maximum total plastic energy dissipation (Figure 8d) in structure and the number of plastic hinges (Figure 8e) , shown in Table 5, increased. At an initial glance, this increase appears undesirable but according to Figure 8e, increasing the number of plastic hinges and the mean maximum plastic energy that accumulated under 22 earthquake acceleration in a plastic hinge in structure (Figure 9e) result in a better distribution of energy in the structure.

It appears that structural damage is distributed in the height of the structure. For instance, the mean maximum plastic energy, which is wasted in a plastic hinge of PBPD structure by considering the SSI effect, is equal to 225.3 kJ, whereas in the structure designed by MPBPD method with SSI effect, the value reaches 204.4 kJ. This implies a 9.3% decrease in the mean maximum plastic energy in a hinge of structure. The rate of decrease is directly related to the decrease in the maximum inter-story drift and displacement in the upper floors of the structure (Figures 8a and 8c). Reduction in acceleration in the fifteenth upper floors (Figure 8b) led to the reduction in non-structural damage that is very valuable and vital for reducing losses due to earthquake. Inter-story drift distribution in the height of the structure is more uniform in the MPBPD case than that in the case of PBPD. Figure 8f shows that shear force declined in all stories of structure with SD less than the PBPD structure. Figure 9 demonstrates the variations of the mean maximum studied parameters with and without SSI effect cases in order to indicate the importance of SSI effect in the design process. According to Figure 9a and 9c, the displacement and inter-story drift without SSI effect is less than that with SSI effect. Although changes in acceleration are less (Figure 9b), total plastic energy dissipated in two cases with and without SSI effect in MPBPD structure show about 3.5 percent decrease and this amount is about 7 percent in PBPD structure (Figure 9d).

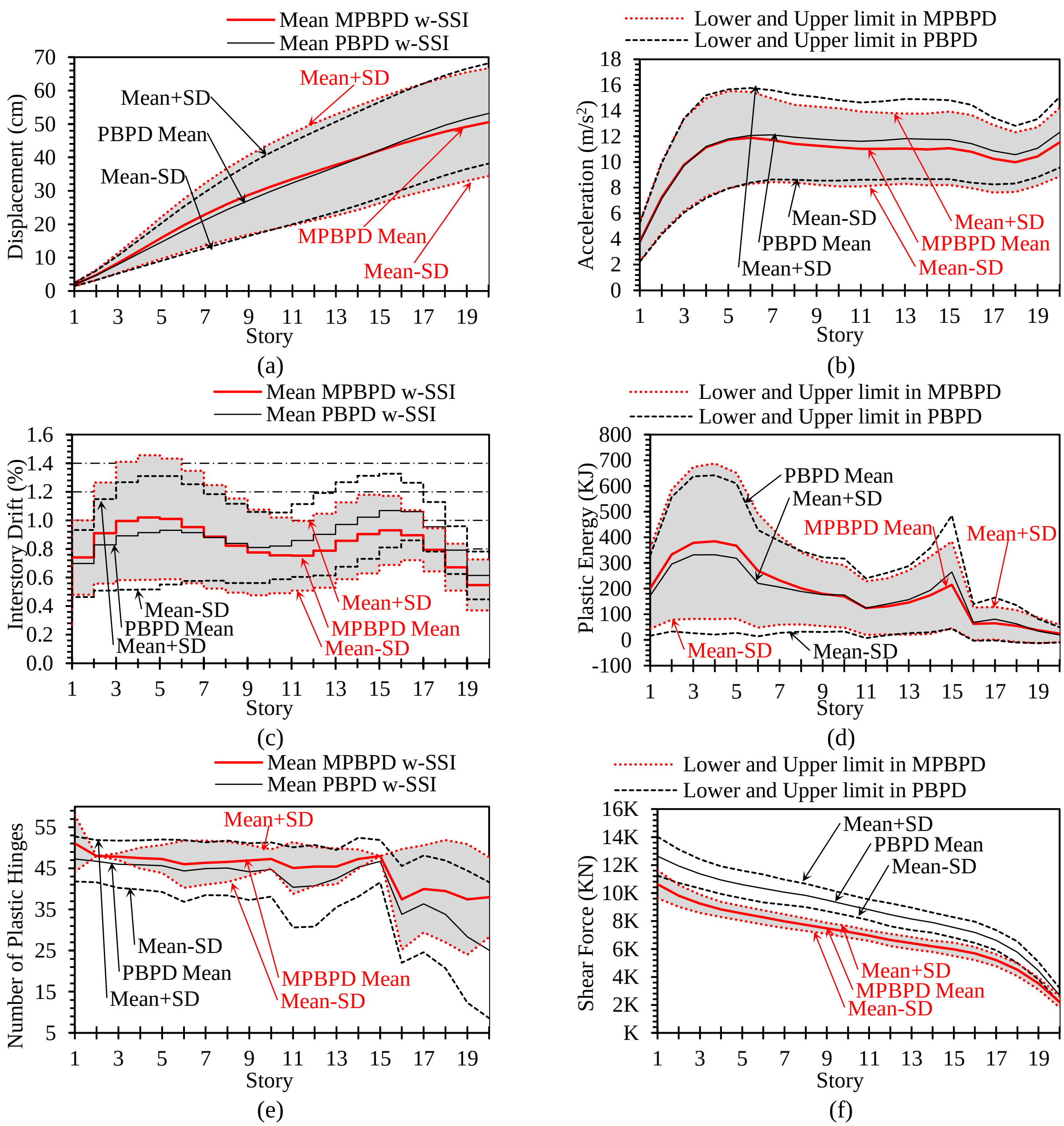


Fig. 8. Mean and standard deviation (μ ± SD, where μ is the mean) of: a) Displacement, b) Acceleration, c) Inter-story drift, d) Plastic energy dissipation, e) Number of plastic hinges, f) Shear force of structure under earthquake excitations

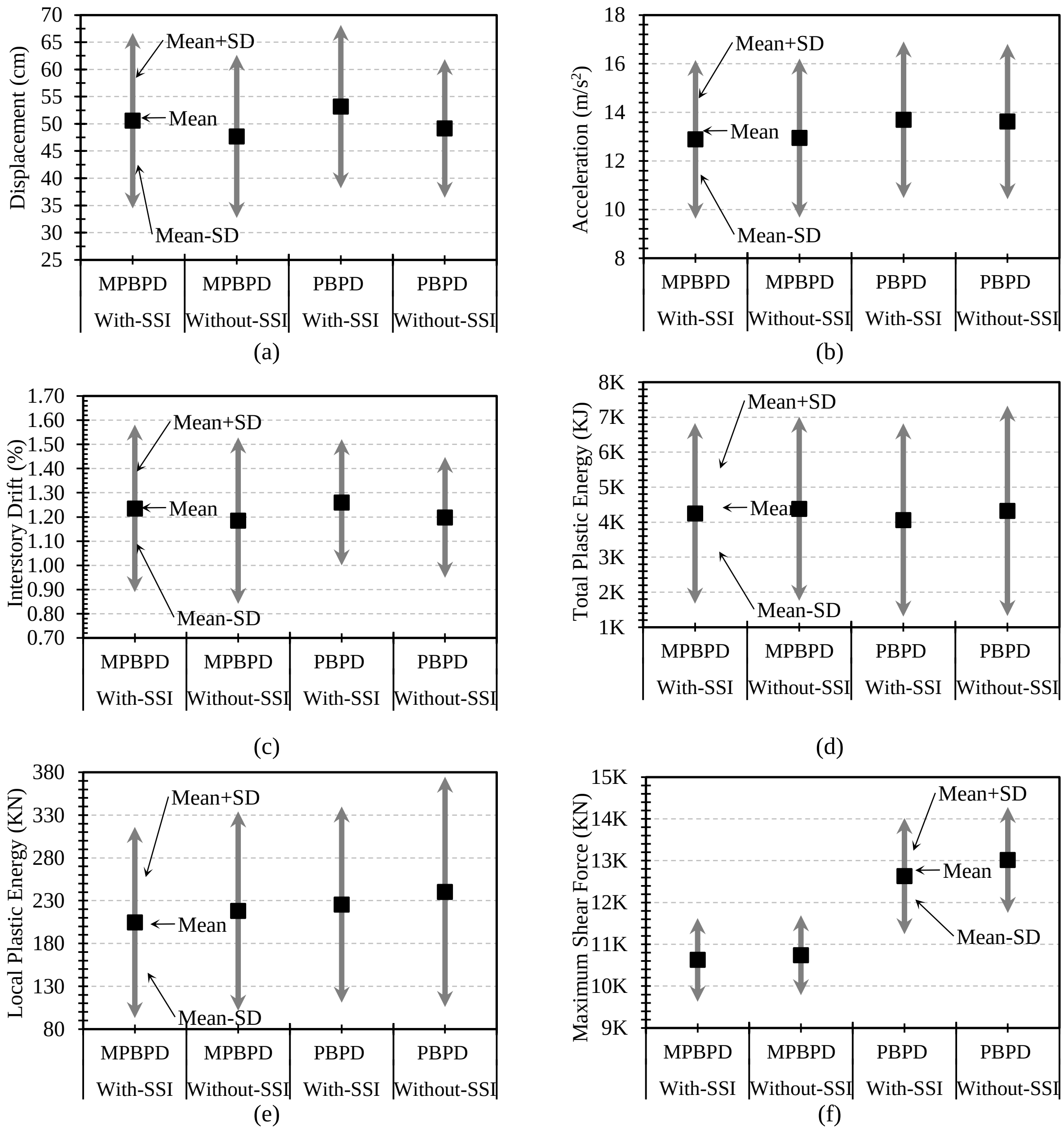


Fig. 9. Mean and standard deviation (μ ± SD, where μ is the mean) for MPBPD and PBPD method in two cases, with and without SSI effect of maximum: a) Displacement, b) Acceleration, c) Inter-story Drift, d) Total plastic energy dissipation, e) Plastic energy that was dissipated in a plastic hinge in structure, f) Shear force of structure under earthquake excitations

Table 5. Compare the maximum amount of the factors affecting the performance of the structure

Parameter	**PBPD Method (with SSI effect)**	**MPBPD Method (with SSI effect)**	**Decrease Percentage**	**Increase Percentage**
lateral displacement (cm)	53.2	50.6	4.9	-
Acceleration (m/s^2)	13.7	12.9	5.9	-
Inter-story drift (%)	1.3	1.2	2.0	-
Total plastic energy dissipation (KJ)	3562.9	3749.9	-	5.3
Number of plastic hinges	827.8	898.3	-	8.5
Maximum shear force (KN)	12630.7	10626.4	15.9	-

As it is evident from Figure 9f, the SSI effect in reducing the amount of base shear is low in a certain design method, but it is actually significantly different between the two methods (MPBPD vs PBPD). Consequently, it should be considered that the SSI effect can have different performance on the structure when compared with predicted values and recommended values considered at the design stage.

Another objective is the nonlinear seismic analysis of the indirect calculation of non-elastic deformation to evaluate the structural efficiency level. Thus, the structural performance levels represent the structural state after being exposed to a certain level of risk.

The FEMA code divides these performance levels into five categories, including:

First level: Full service capacity for maximum Inter-story drift less than 0.2.

Second Level: Service capacity for a maximum Inter-story drift of less than 0.5.

Third level: Physical safety for a maximum Inter-story drift of less than 1.5.

Fourth level: The probability of structural collapse for a maximum Inter-story drift of less than 2.5.

Fifth level: the destroyed structure is for the maximum inter-story drift more than 2.5.

In Figure 10, the earthquakes under which the structure crossed the physical safety is marked with a red circle. Regarding the results obtained in Figure 10, it is clear that considering the effect of SSI in analyzing nonlinear time histories, the PBPD method has a clear vulnerability. The number of earthquakes causing the structure to pass through the boundary of the level of safety increase twice. Although in the design of MPBPD, the inter-story drift has increased significantly, the number of earthquakes that cause the structure to cross the boundaries of the physical safety level has declined. As a result, the design of the MPBPD has led to an increase in the efficiency of the structure.

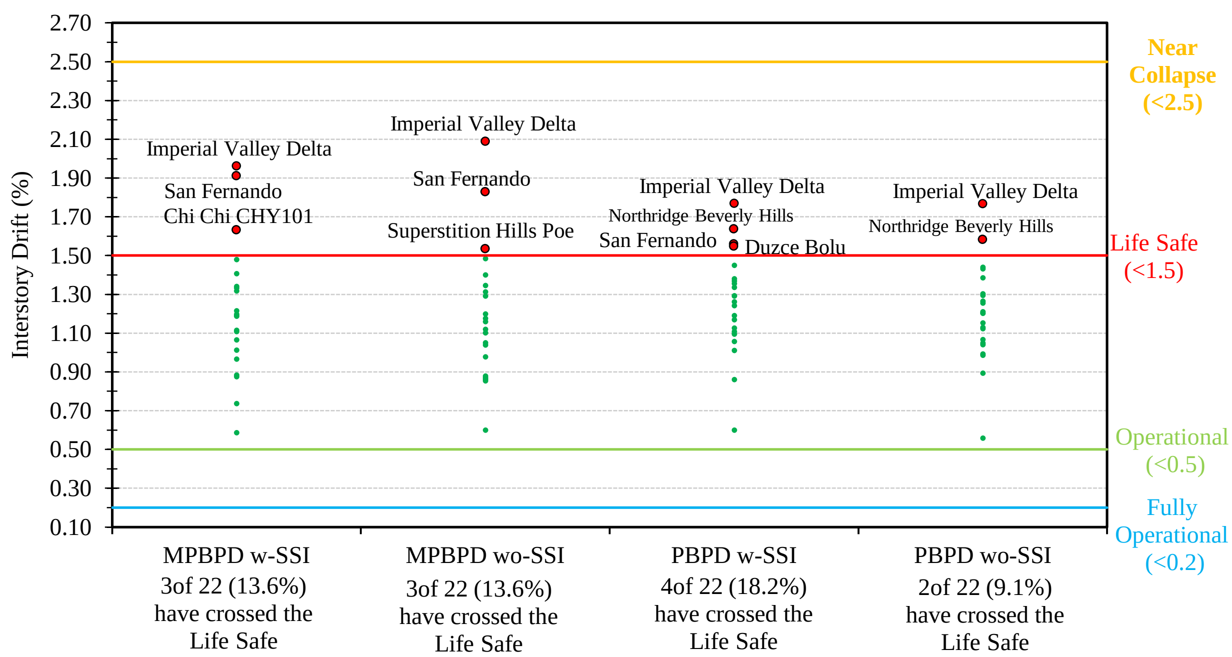


Fig. 10. Investigating structural performance levels in four different modes

CONCLUSIONS

In this research, by applying a complete wealth of far-field earthquake records, two methods of MPBPD and PBPD have been evaluated precisely for seismicity. Numerical simulations are done on three-dimensional finite element model for twenty-story concrete structures. The result reveal that, with the involvement of the interaction effect in PBPD design method, significant changes in the response of the structure is created that cannot be ignored. By examining six seismic responses related to structural performance, the superiority of the proposed design method is well established compared to the conventional design of PBPD. By analyzing the maximum inter-story drift values and comparing them in different states of structural analysis, it has been proven that the design of MPBPD results in an increase in the structural efficiency. In the MPBPD design method, maximum displacement, acceleration, inter-story drift and shear force dropped leading to a better distribution of energy in the structure compared to the PBPD method. MPBPD method is recommended to practicing engineers in order to ensure the safety of high- rise buildings in terms of seismic design and soil-structure interaction in high-risk regions.

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