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Considering the Soil Effects on Design Process of Performance-Based Plastic Design for Reinforced Concrete Structures

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ABSTRACT: In this research, Performance-Based Plastic Design (PBPD) method has been modified according to the proposed method for considering Soil–Structure Interaction (SSI) effects. In the proposed modified method, based on the existing relationships and in order to maintain the simplicity of the PBPD design method, two important parameters have been modified in the PBPD design method. These two parameters include the modification of the vibration period of the structure due to the effect of SSI and the lateral target displacement modification, which is a key parameter in the evaluation of the structural performance. Efforts have been made to refine the modifications to maintain the simplicity and robustness of the PBPD equations. Finally, design base shear force of the PBPD method has been corrected due to the SSI effect regarding the modified relationships. By making the modifications, in order to better understand this method, structures with a number of different floors, including the 4, 8, 12, and 20 spatial space moment frames, are designed and compared with the results of designing the method without the effect of SSI and design method based on the capacity.

Keywords: Concrete Structures, Inelastic Behavior, Moment Frames, Performance-Based Plastic Design, Soil–Structure Interaction.

INTRODUCTION

The dominant method of designing structures under seismic loads in most countries, even if it is known that the structure undergoing intense earthquakes experiences large deformations in the non-elastic boundary, is carried out elastically and the nonlinear behavior of the structure is indirectly considered. Numerous studies show that this design process is not suitable for preventing breakdown mechanisms (Wongpakdee and Leelataviwat, 2014, 2017). For example, previous research has well established that the distribution of earthquake lateral force at a structural altitude based on the elastic behavior of structures, which is carried out in terms of force-based design, is significantly different from the results of the analysis of nonlinear time histories (Chao et al., 2007). Verde (1991) correctly proved that the distribution of lateral forces at the height of the structure, regardless of the fact that the structure under intense earthquakes comes

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into the nonlinear zone, is the main reason that caused most of the upper floors to collapse during the Mexico city earthquake of 1985. As another example of the carried out research, Choi et al. (2013) also pointed out that with the use of a stiffness ratio between columns and structural beams, which is often a ratio of 2 to 1 , there are attempts to prevent the occurrence of plastic joints in the columns, which unfortunately, in practice, is due to lower estimates of the column moments in the plastic structure joints. In general, the design weaknesses of this design process include non-steady-state control, severe submission, buckling, rupture, and local instability of the structural members that can occur widely and non-uniformly in the structure and eventually result in an unpredictable and undesirable response followed by the overall rupture of the structure (Goel and Chao, 2008; Liao, 2010; Sahoo and Chao, 2010). Considering the weaknesses in the force-based design, in the past decade, there has been a significant tendency and direction to design based on the performance of structures in the engineering community. In this respect, various methods have been developed, among which are the Capacity Spectrum Method (Riga et al., 2017), the N-2 method (Fischinger, 2014), the Yielding point Spectrum Method (Thermou et al., 2012), the Modified Lateral Force Method (Calugaru and Panagiotou, 2012), the Direct Displacement -Based Design Method (Priestley et al., 2007) and Performance-Based Plastic Design (Goel and Chao, 2008; Abdollahzadeh et al. 2017). In the vast majority of these methods, attempts have been made to revise the determination of design base shear force by considering the effects of higher modes, structural strength addition, variations of yielding displacement, effective stiffness, viscous damping, effective period or structural ductility. Among all these methods, the PBPD design method has been found to be significant among researchers, and it seems that this method can be a logical alternative to the traditional incorrect seismic method of the structure, based on force. In this method, contrary to the current methods in the design guidelines of design base shear force for a chosen level of risk equating the work required to deliver the structure to the lateral target displacement uniformly, with the energy required in a single-degree-of-freedom (SDOF). In this method, a structure will be produced that will behave based on performance limit cases such as lateral target displacement and the desired yielding mechanism. In this method, the nonlinear behavior of the structure is considered directly and essentially eliminates the need for evaluation and repetition by static nonlinear analysis or time history analysis after the initial design. The full details, the design method of PBPD, as well as the proof of the governing theory, can be found in the design guide provided by Goel and Chao (2008).

Despite all the advantages and implications of the design method of PBPD, the point that has so far not been taken into account is considering the soil as the main base for the transfer of seismic vibrations to the structure. Many construction design codes suggest that the SSI effects can be ignored and the structure base can be considered fixed (Mylonakis and Gazetas, 2000; ASCE, 2010; Li et al., 2014). While research on the effects of neglecting SSI has shown that the presence of soil under the foundation leads to the establishment of lateral and rotational degrees of freedom in the foundation and can significantly increase the demand for ductility. It can also change the vibration period of the structure, the damping ratio and the mode shapes which, in turn, can lead to an increase or decrease of nonlinear behavior in the structure system, and a change in the overall behavior of the soil and structure system, and consequently a change in the system response, including increased lateral displacement and inter-story drift under earthquake loading, and ultimately leads to faulting and eliminating performance criteria expected from the structure. There have been several cases of severe damages in structures due to SSI in the past earthquakes (Chinmayi and Jayalekshmi, 2013; Tabatabaiefar et al., 2014; Morshedifard and Eskandari-Ghadi, 2017). Mylonakis et al. (2006) carried out several analyses and have concluded that SSI is one of the reasons behind the collapse of Hanshin Expressway in 1995 Kobe earthquake and shown that increase in natural period of structure due to SSI is not always beneficial as suggested by the simplified design spectrums in design codes.

In this research, with the involvement of soil as the main base of vibration transmission to the structure, the design of the PBPD was deployed from a design method solely for the design of the structure for the soil-foundation and the structure system. For this purpose, with the modification of two key parameters in the PBPD design method, structural vibration period and the lateral target displacement of the structure, the PBPD design method has been modified to consider the soil effect. In order to understand the proposed method, four special RC frames used in the FEMA P695 (Agency, 2009) have been redesigned and compared with the results of the design in the PBPD method without the effect of SSI and the force-based design method. In order to verify and compare the results fairly, except for the design method, other assumptions intended to redesign the frames are exactly consistent with the parameters and assumptions used in FEMA P695.

DESIGN PROCEDURE

The overall design process of the PBPD approach, as described below, helps to build an ideal implementation structure by considering the issues that are more common among designers. One of the important issues of designers interest in structural design and can be achieved within the framework of the PBPD design method, is increasing the lateral displacement control of the structure, increasing safety levels, saving the amount and cost of materials in the construction of the structure and ease of numerical calculations. The design steps according to this method can be so stated that, in the first stage, a yielding mechanism and a specific value for the lateral target displacement of the structure are selected. The vibration period of the structure is estimated based on the empirical relationships existing in the technical literature or construction codes. In the second stage, the design base shear force for the structure is obtained at a certain level of risk by equating the energy required for the structure to reach the lateral target displacement and the equivalent energy in a structure of one degree of freedom. It is worth noting that if the behavior of structural materials does not comply with the elasto-plastic behavior, as the concrete behavior in reinforced concrete structures, the design base shear force should be modified. In the last stage, the members selected in the yielding mechanism are subjected to nonlinear behavior are designed based on the plastic design method, and other members behaving elastically in the yielding mechanism will be designed according to the capacity method. This method automatically reduces the uncertainties of the frames with a high degree of uncertainty, so that all design and analysis stages can be done easily with the hand, and it also helps engineers to better understand the behavior of the structure under seismic loading (Goel and Chao, 2008). The PBPD methodology reminds us of the portal method in the frame analysis, but the governing processes in terms of both the nature and the thought in two methods are completely different and distinct from each other. The simple and basic assumption in the portal method is that the inflection points are formed in the middle of the beams and the height of the structural columns, and essentially the portal method is, not a design method, an approximate analysis method for the analysis of structures. On the other hand, in PBPD method, which is a design philosophy, the linear variations of the relative displacement of the structure are chosen so that, during the linear and nonlinear behavior of the structure, the inflection points are forced to form at the points for which the structure is designed. Promising and innovative, the PBPD design method paves the way for designing more economical, secure, and reliable systems in the future.

The Current Trend of PBPD Method to Determine the Design Basic Shear Force without Soil Effect

The basic shear force is obtained in a PBPD-based plastic design at a certain level of risk based on the non-elastic state of the structure, which is the desirable yielding mechanism, controlling the relative displacement of the structure. For this reason, there is no need for a separate control of relative displacement after the design. The basic shear force design is determined by bringing the structure to the relative target displacement after formulating a favorable yielding mechanism that has been selected by the designer. There is no need to do any kind of pushover analysis at the stage of determining the basic shear force and at other stages. The use of the energy equation in a simplified manner with the design of the limit state was first suggested by Housner (1956), who used the difference between the energy input to the structure and the elastic strain energy to reach the amount of plastic energy. This amount of plastic energy is the same used to design yielding members. Housner (1956) proved that the velocity response spectrum for earthquakes that are commonly used in the analysis of seismicity of the structure in a wide range of vibration periods proved to be constant in order to design the seismicity of structures. If we show the pseudoacceleration with S_a, we will have:

$$S_a = a.g \quad (1)$$

in which, a: is the normalized pseudoacceleration, resulted from the division of the pseudoacceleration into the gravity acceleration g. The pseudivelocity, we will show with S_v, from elastic response spectrum will be equal to:

$$S_v = \frac{S_a}{\omega} = \frac{T}{2\pi}.S_a = \frac{T}{2\pi}.a.g \quad (2)$$

where T: is the main period of the structure. If M is the total seismic mass of the structure, the weight of the structure (W) is obtained in accordance with Eq. (3).

$$W = M.g \quad (3)$$

As a result, the equilibrium of the total energy of the structure is obtained using Eq. (4), assuming that the energy required to convey the structure uniformly to the maximum displacement is equal to the maximum energy input of the earthquake from an elastic system.

$$\begin{aligned} E_{total} &= \frac{1}{2}M.S_v^2 = \frac{1}{2}M.\frac{T^2}{4\pi^2}a^2g^2 \\ &= \frac{1}{8\pi^2}T^2a^2gW = \frac{Wga^2T^2}{8\pi^2} \end{aligned} \quad (4)$$

Since the proposed equation was obtained by considering some assumptions, the researchers sought to modify Eq. (4) and concluded that the input energy was equal to the multiplication of the Housner equation (Housner, 1956, 1959; Kato and Akiyama, 1982; Akiyama, 1985). This coefficient is called energy modification coefficient and is indicated by γ. As a result, the input energy equation is as follows.

$$\mathrm{E_{total}} = \gamma\left(\frac{1}{2}\mathrm{M.S_v^2}\right) = \frac{1}{2}\gamma\mathrm{M}\left(\frac{\mathrm{T}}{2\pi}.\mathrm{S_a}.\mathrm{g}\right)^2 ; \quad \text{where} \rightarrow \ \gamma = \frac{2\mu_s \text{-}1}{R_\mu^2} \tag{5}$$

where μ_s: is the coefficient of ductility and R_μ: is the coefficient of structural ductility reduction. Another important component of the equilibrium equation is the elastic energy (E_e). For this kind of energy, Kato and Akiyama (Kato and Akiyama, 1982) showed that this energy can be calculated using Eq. (6), assuming that the entire structure is considered as a structure of a degree of freedom with an acceptable accuracy.

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

in which V_y: is the yielding base shear. Based on Eqs. (5) and (6), the total energy of plastic (Eq. (7)), which the structure wastes during an earthquake, is obtained by subtracting two Eqs. (5) and (6):

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

The distribution of design lateral forces in the current construction codes is based on the elastic response of the main mode of vibration of multi-degree of freedom systems, and has recently been revised to take into account the effect of higher vibrational modes. Chao et al. (2007) presented Eqs. (8-10) for the distribution of lateral forces based on the relative distribution of maximum shear force, which is in good agreement with the results of non-elastic dynamic response. Also, in this equation, the effect of higher modes is also considered.

$$F_i = C'_{vi}V \tag{8}$$

$$C'_{vi} = \left(\beta_i - \beta_{i+1}\right)\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} ; \tag{9}$$

$$i=n \quad \beta_{n+1}=0$$

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{j=i}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} \tag{10}$$

The constants of Eqs. (8-10) are empirically obtained by fitting the results of the non-austenitic response to a wide range of commonly used structural systems (Chao et al., 2007). In the above equations for $i=n$, the shear V_n and the lateral force F_n: are obtained on the roof floor. β_i: is the shear distribution coefficient at the level of the i-th floor. V_i and V_n, respectively, show the shear forces of the floor at the level of i and at the highest level (the roof floor). W_j: is the seismic weight in the j floor, h_j is the height of the j floor from the foundation level, W_n: is the weight of the roof, h_n the height of the roof floor from the base level, T: is the natural oscillation period of the structure, F_i is the lateral force of the floors i and V, the total base shear force of the design.

Using the desired yielding mechanism for a specific structural system, as shown in Figure 1a, and equating the plastic energy (E_p) and external work performed by distributed lateral forces (Eq. (11)), the amount of base shear force of the design according to Eq. (12) is easily achieved.

$$E_p = \sum_{i=1}^{n} F_i h_i \theta_p \tag{11}$$

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}}{2} \tag{12}$$

where V_y: is the base shear force of the design, and α is a dimensionless parameter that depends on the amount of plastic displacement, the stiffness of the structure, and the modulus properties of the structure, based on Eq. (13).

$$\alpha = \left(\sum_{i=1}^{n}(\beta_i - \beta_{i+1})h_i\right). \left(\frac{w_n h_n}{\sum_{i=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}} . \left(\frac{\theta_p 8\pi^2}{T^2 g}\right) \tag{13}$$

in Eq. (13), β_i: is the shear distribution coefficient in the i-th floor level, θ_p: is the relative displacement ratio of plastic, W_j: is the seismic mass in the j floor, h_j: is the height of the j floor from the foundation, W_n is the weight of the roof, h_n: is the height of the roof floor from the base and T: is the period is the natural oscillation period of the structure. In Eq. (13), when $i=n$, $\beta_n+1=0$ will be considered. In Eq. (12), lateral target displacement of the design is also considered, thereby controlling relative displacement at the beginning of the design stages.

CONSIDERATIONS SSI EFFECTS IN PBPD METHOD

In PBPD method, two basic parameters, the relative target displacement of the design and selective yielding mechanism, play a fundamental role in the implementation of this design method. If a fundamental factor such as soil is not considered in the design process, it can disrupt these two critical parameters and easily shed doubt on the entire design process. As discussed earlier, the results of the analysis of integrated soil-foundation and structure system, three changes with soil considerations are mentioned, which include: longer period of the structure, the modification of the shape of the structures and the damping of the structure.

It is clear that of three changes due to the effect of soil only the change in vibration period in the design process is dependent on the PBPD method, and other changes do not have any effect on this method. But as we know and previous studies have paid less attention to it, the other effect of the soil that occurs when the soil is yielded is to increase relative displacement of the entire structure due to the rotation of the foundation.

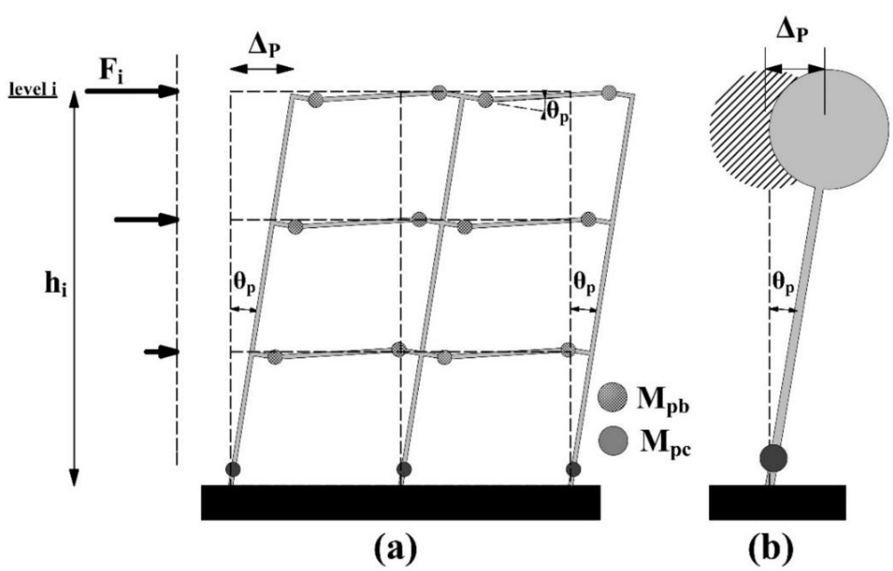


Fig. 1. a) The desired yielding mechanism of reinforcement concrete spatial moment frames, b) The equivalent single degree of freedom

Therefore, if, in addition to the vibration period of the structure, a solution is adopted to modify the relative displacement of the design due to soil yielding, then the PBPD design method can be modified to consider the soil effect as shown in Figure 2. It is worth noting that apart from the lateral load distribution equation in the PBPD design method proposed by Chao et al. (2007), other relationships have been obtained over 60 years of research, and by combining these equations together, the method of PBPD design presented by Goel and Chao (2008) can be considered. So, with these reforms, one can take a step forward in developing a modern and up-to-date design approach and understand the changes made in the effect of the soil in a tangible way.

Modification of Structure Vibration Period under the Soil Effect

There are numerous relationships for estimating the vibration period of the structure due to the effect of SSI on technical literature and building codes. Since the maintenance of simplicity and ease of numerical calculations has been the objective of this research, in this research the familiar equation of ASCE 7-10 code, Eq. (14), is used to calculate the structure vibration period by considering the effect of SSI.

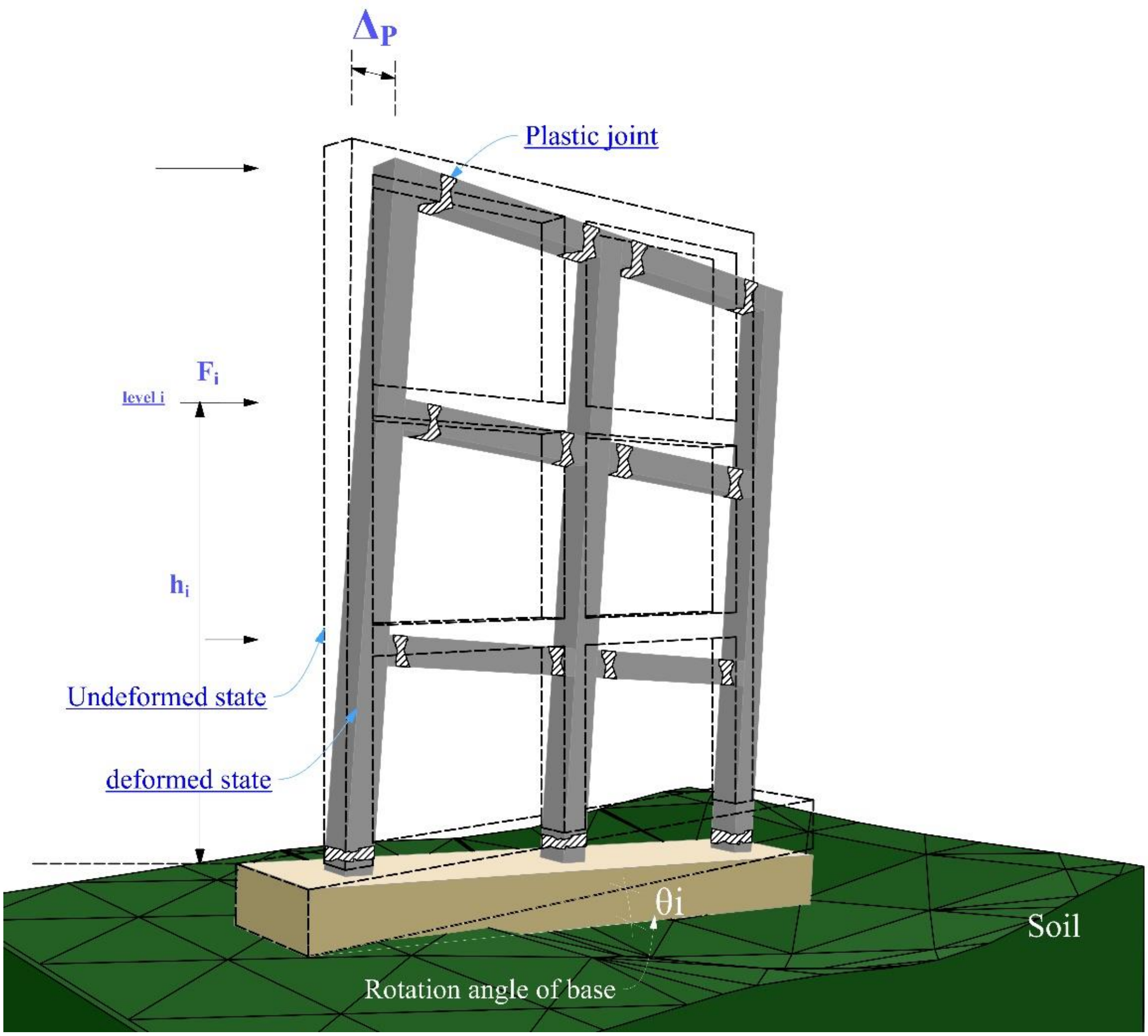


Fig. 2. The SSI effect in PBPD method

It is worth mentioning that in the current design of PBPD, the vibration period is obtained from the equation expressed in this code without the effect of SSI.

$$\tilde{T} = T\sqrt{1 + 25\alpha \frac{r_a \bar{h}}{V_s^2 T^2}\left(1 + 1.12\frac{r_a h^2}{\alpha_\theta r_m^3}\right)} \quad (14)$$

where α: is relative weight density of structure and soil can be determined as below:

$$\alpha = \frac{W}{\gamma A_o \bar{h}} \quad (15)$$

r_a and r_m: are characteristic foundation lengths and are defined by:

$$r_a = \sqrt{\frac{A_o}{\pi}} \quad and \quad r_m = \sqrt[4]{\frac{4I_o}{\pi}} \quad (16)$$

T: is fundamental period; α_θ: is dynamic foundation stiffness modifier for rocking motion effects; $\bar{h}$: is effective height; V_s: is shear wave velocity; A_0: is the area of the load-carrying foundation and I_0: is the static moment of inertia of the load-carrying foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed.

Modification of Relative Target Displacement due to Soil Yielding

In order to estimate the relative displacement variations added due to the soil yielding and the foundation rotation due to soil compaction, after extensive investigations, Eq. (17) proposed by Poulos and Davis (1980) has been used for this purpose. It is worth noting that most relations in the technical literature that measure the foundation rotation have relatively complex items that require the simultaneous analysis of the structure and are virtually impossible to use in the design process.

$$\tan\theta_i = \frac{1-\nu^2}{E}\cdot\frac{M}{B^2.L}.I_\theta \quad (17)$$

wher θ_i, the rotational angle of the foundation, is due to the soil yielding. M denotes the amount of bending moment at the base of the column. ν, E and I_θ: are parameters related to the type of soil the structure is based on.

Modified Relationships to Consider the Soil Effect

After modification of the vibration period of the structure and determination of the amount of relative displacement changes added, due to the foundation rotation, the following relationships are proposed to improve the design process. By modifying the vibration period of the structure, μ_s, the coefficient of ductility and R_μ, the structural ductility coefficient, both of which depend on the amount of vibration period of the structure, and directly determine the amount of energy modification coefficient, are modified based on proposed Eqs. (18) and (19).

$$\mu_s^* = \frac{\Delta_{\max}^*}{\Delta_y} \quad (18)$$

$$R_\mu^* = \frac{\Delta_{eu}^*}{\Delta_y} \quad (19)$$

in which, μ_s^* and R_μ^*: are the modified values of the coefficient of ductility and the coefficient of structural ductility reduction due to the effect of SSI. Δ_y: is the amount of displacement of the structural yielding, Δ_{max}^*: is the maximum displacement of the structure due to the effect of the soil and structure interaction, Δ_{eu}^*: is the final displacement of the structure, with the assumption of its elastic behavior. As a result, the amount of energy modification coefficient will be equal to:

$$\gamma^* = \frac{2\mu_s^* \text{-}1}{(R_\mu^*)^2} \tag{20}$$

In order to modify the target displacement of design, the amount of rotation angle obtained is summed with the proposed value in the conventional PBPD method. As a result, we will have:

$$\theta_u^* = \theta_u + \theta_i \tag{21}$$

where θ_u^*: is the relative target displacement determined by considering the effect of the soil. Eqs. (6-7) and (12-13) are modified with the modified relationships and the remaining steps in the design process will be performed according to the conventional method.

DESIGN EXAMPLES

The 4, 8, 12 and 20-story space frame buildings used in the FEMA P695 were redesigned with consideration of soil–structure interaction (SSI) effect as discussed earlier. The 3D floor plan and elevation of structures are shown in Figures 3 and 4. Bay width in 8, 12, 20 story is similar and equal to 20 feet (6.09 m). In the 4-story case, bay width is 30 feet (9.14 m). The 28-day compressive strength of concrete for structural columns and foundation is equal to 7 ksi (42 MPA) and for beams is 5 ksi (35 Mpa). The yielding stress of steal is considered to be 60 psi (414 MPa). Design of floor dead and live load respectively, is 175 psf (854 kg/m^2) and 50 psf (244 kg/m^2). For all designs, P-Delta effect was accounted for by using a combination of gravity loads on the moment frame. In terms of building site, the moment frame was designed for a general high seismic site in Los Angeles, California (soil floors S_d, S_{ms}=1.5 g and S_{m1}=0.9 g). According to foundation design method, for each RC SMF, Rigid strip foundation was considered as the following Table 1. Beams are considered as Designated Yielding Members (DYM). Beam flexural capacity required on each floor, with plastic design approach and with respect to Figures 1.a and 1.b, is determined using the following equation:

$$\sum_{i=1}^{n} F_i h_i \theta = 2M_{pc}\theta + \sum_{i=1}^{n} 2\left(\beta_i M_{pb}\right)\gamma_i \tag{22}$$

where θ: represents rotation angle of the yield mechanism, M_{pb} and $\beta_i M_{pb}$: are the required moment strengths at the top floor level and level i, respectively, and γ_i: is rotation term. The columns that are required to remain elastic are designed based on a capacity design method.

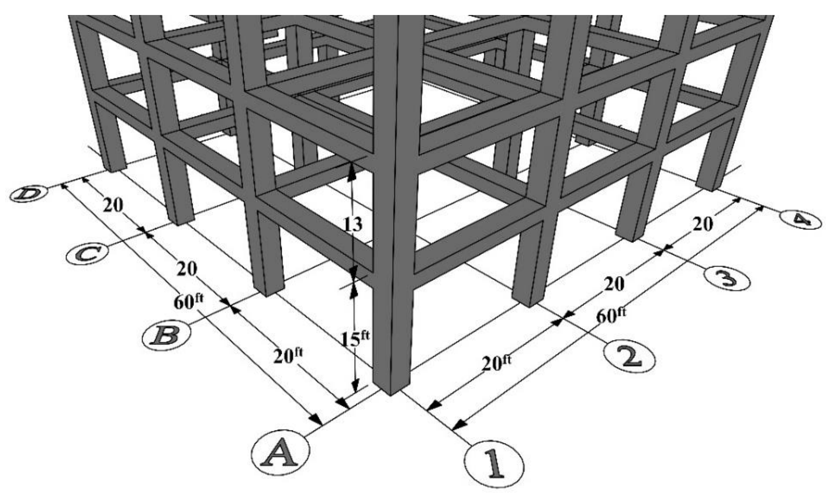


Fig. 3. 3D Floor plan of design examples

Table 1. The Specification for strip foundation

Building	Dimension Height ft (m)	Width ft (m)	Length ft (m)	Compressive Strength Concrete of Structure ksi (MPa)
4 story	3 (0.9)	5(1.5)	90 (27.4)	5 (34.5)
8 story	3 (0.9)	5(1.5)	60 (18.3)	5 (34.5)
12 story	4 (1.2)	5(1.5)	60 (18.3)	6 (41.4)
20 story	5 (1.5)	5(1.5)	60 (18.3)	6 (41.4)

DESIGN RESULTS

The main weakness of current seismic design code for RC SMF is lack of guidance to provide the engineers as to how to achieve the desired goals such as, controlling drifts, distribution and extent of inelastic deformation, etc. In contrast, the PBPD method is a direct design method, which requires no evaluation after the initial design because the nonlinear behavior and key performance criteria are built into the design process from the start. The variations of the fundamental period of structures with considering the SSI effect are shown in Table 2. Design process for each frame, in order to comparison was done in three methods: 1) PBPD method result with SSI effect, 2) PBPD method result without SSI effect and 3) strength based method result based on FEMA. Design base shear was obtained with and without SSI effect present in Table 3. Figure 6 is shown the distribution of lateral design force in elevation of structure. In frame elevation of Figures 7-10, dimension of members and reinforcement ratio of 4, 8, 12 and 20-story space frame buildings is shown graphically.

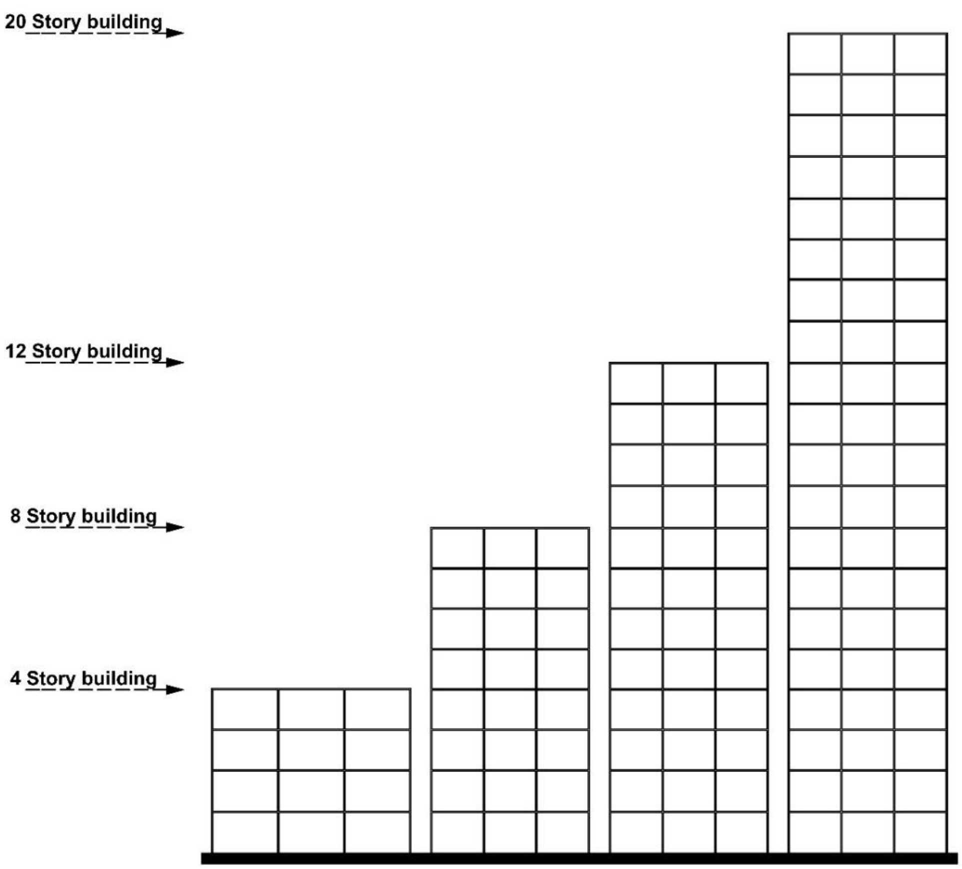


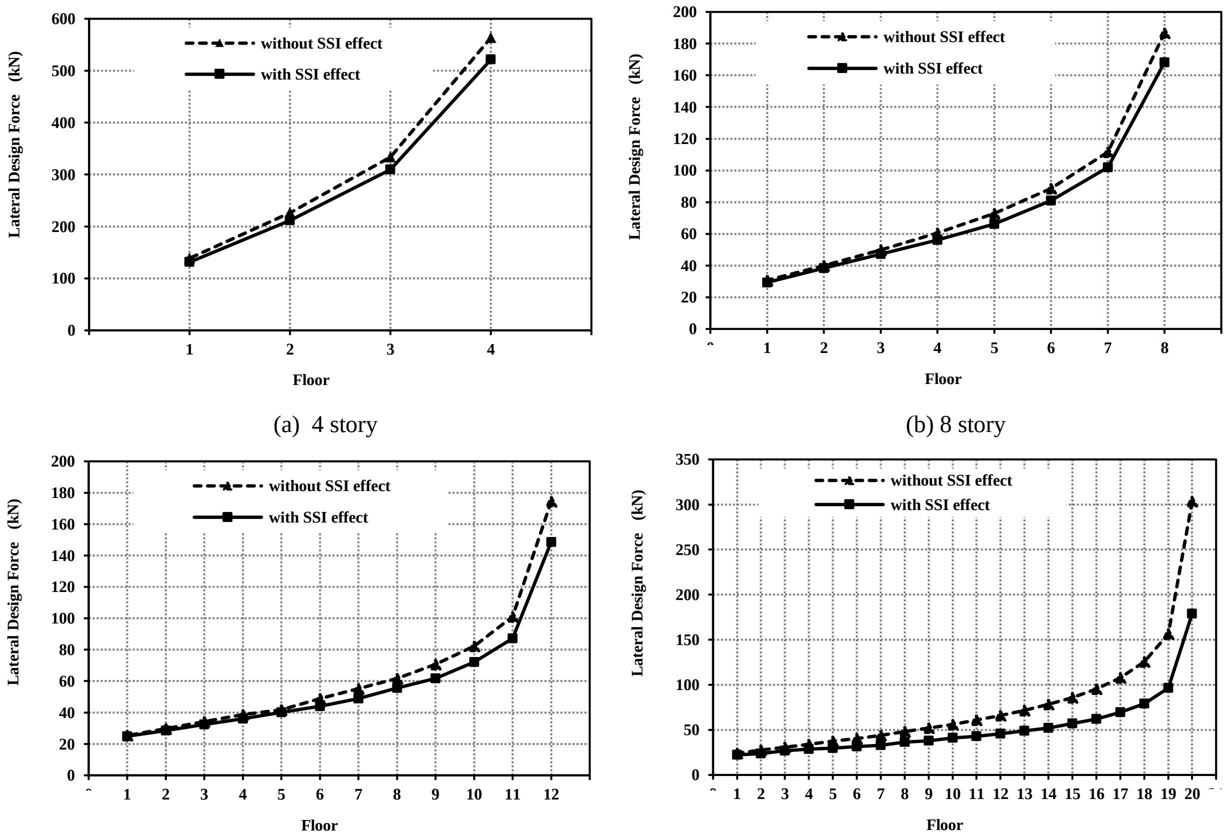
Fig. 4. Elevation of design examples

Table 2. Fundamental period of structure

Building	T (sec) (without SSI Effect)	$\tilde{T}$ (sec) (with SSI Effect)	Increase Percentage
4 story	0.81	0.842	3.8%
8 story	1.49	1.567	4.9%
12 story	2.13	2.307	7.7%
20 story	3.36	3.856	12.9%

Table 3. Compare design base shear (with P-Delta effect)

Building	PBPD Method (without SSI Effect)	PBPD Method (with SSI Effect)	Decrease Percentage
4 story	1262 kN	1173 kN	7.1%
8 story	641 kN	588 kN	8.3%
12 story	764 kN	680 kN	11.5%
20 story	1545 kN	1043 kN	32.5%



(a) 4 story (b) 8 story

(c) 12 story (d) 20 story

Fig. 5. Distributed lateral design force for 4, 8, 12 and 20-story frame buildings

CONCLUSIONS

In this research, PBPD method was modified to consider SSI effect. Proposed procedure is a user friendly method for practical uses. Four RC SMF as used in the FEMA P695 was redesigned in this study. Result of design show that, involving the interaction effect in PBPD design method, distribution of rebar in beam and columns was changed. However, according to the changes in determining the design base shear, it is expected. By decreasing the height of the structure, this change was intensified so that, the approximate rate of reduction in 4-story building is seven percent and in 20-story building is thirty-two percent.

It seems that symmetric plastic joint in selected mechanism of special moment frame, used in PBPD method, leads to handle additional rotation caused by failure of soil.

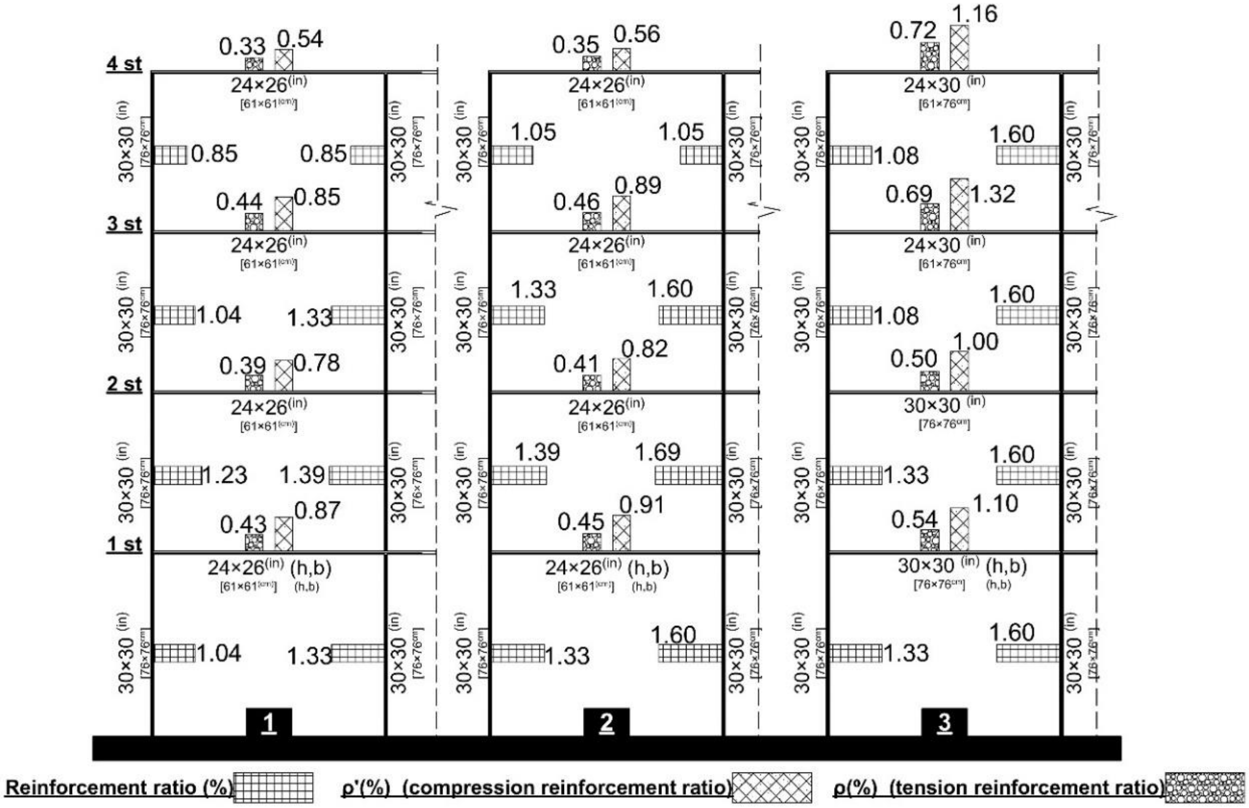


Fig. 6. Detail of design 4 story building in: 1) PBPD method with SSI effect, 2) PBPD method without SSI effect and 3) Strength based method

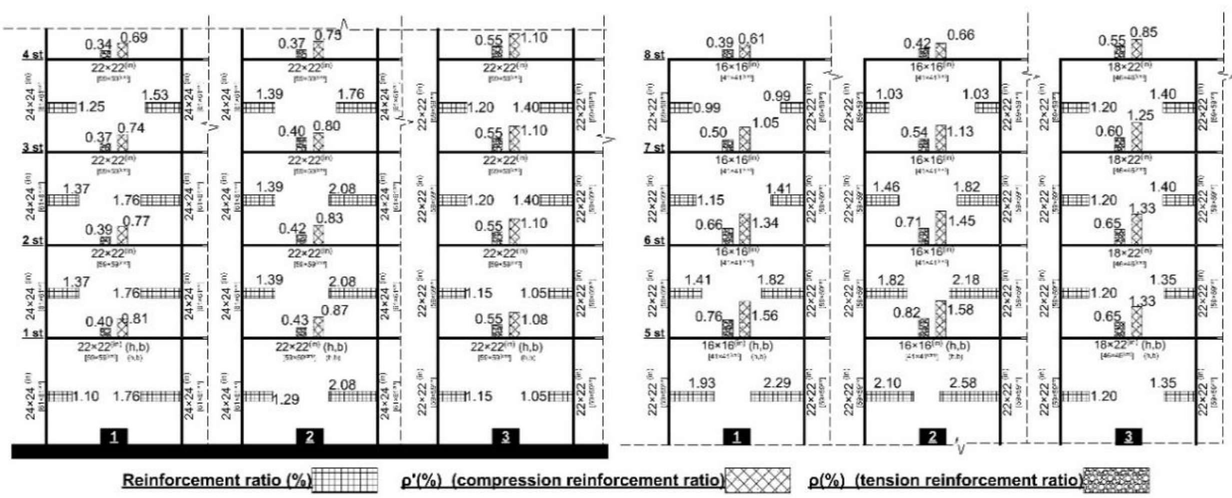


Fig. 7. Detail of design 8 story building in: 1) PBPD method with SSI effect, 2) PBPD method without SSI effect and 3) Strength based method

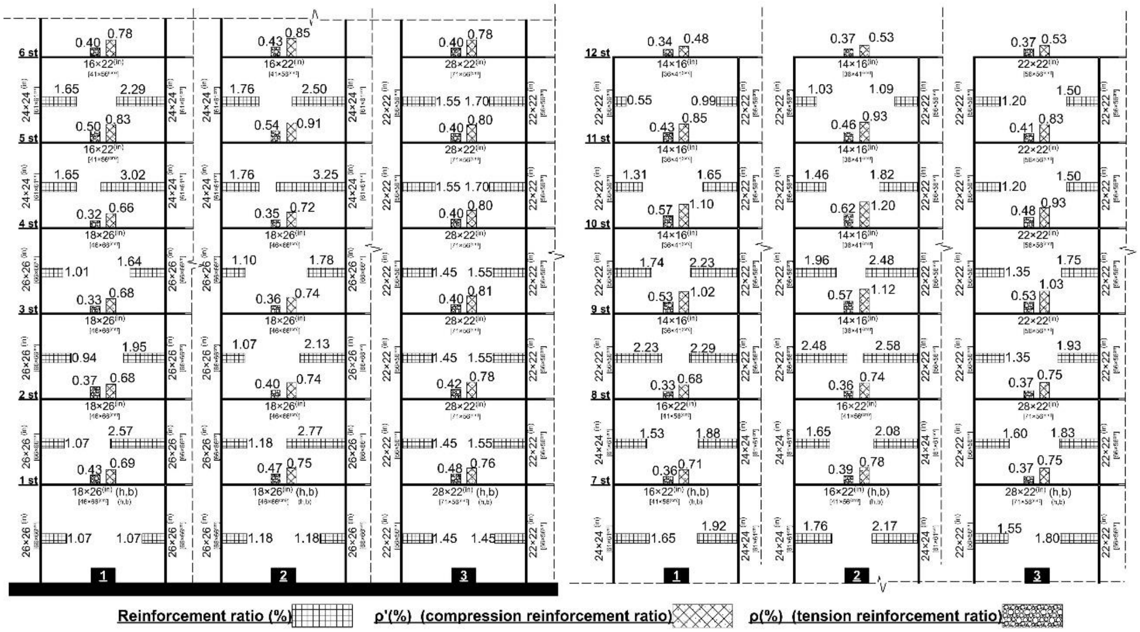


Fig. 8. Detail of design 12 story building in: 1) PBPD method with SSI effect, 2) PBPD method without SSI effect and 3) Strength based method

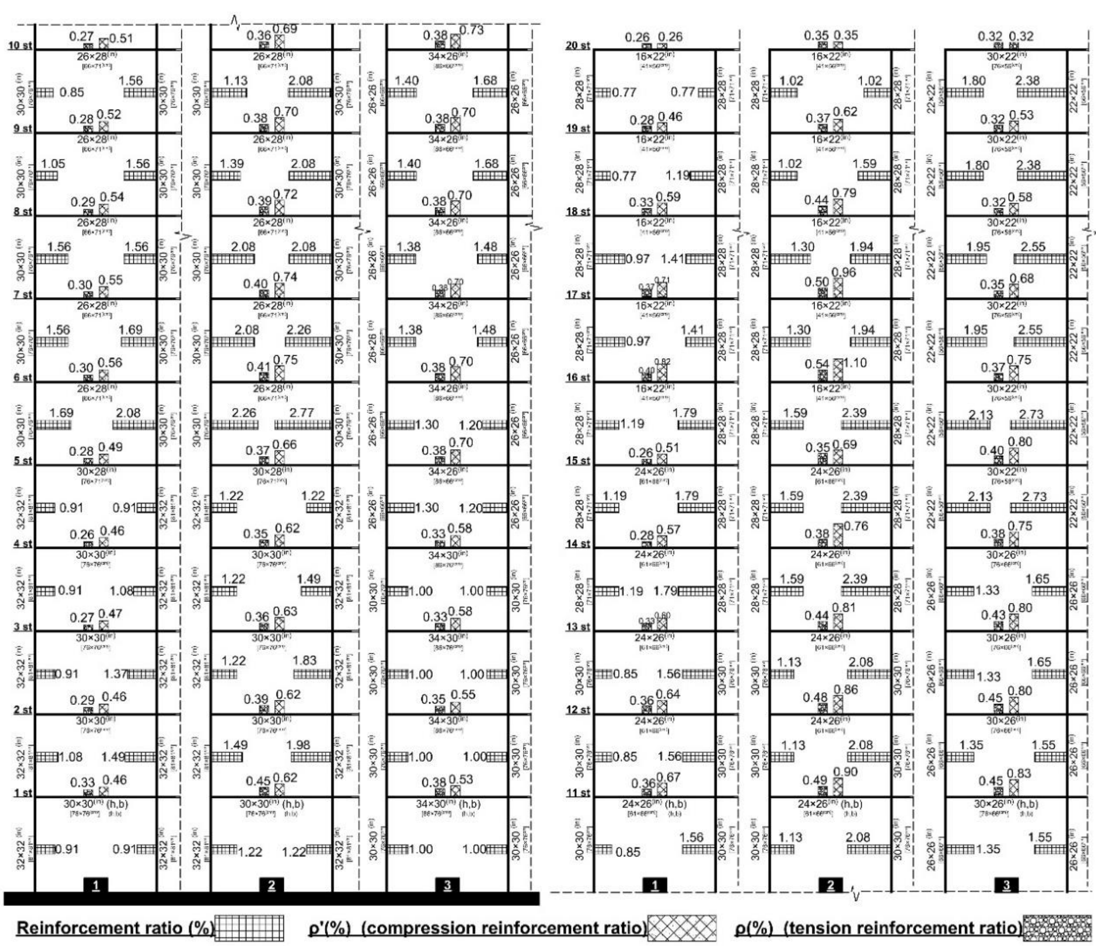


Fig. 9. Detail of design 20 story building in: 1) PBPD method with SSI effect, 2) PBPD method without SSI effect and 3) Strength based method

REFERENCES

American Society of Civil Engineers (ASCE). (2010). *Minimum design loads for buildings and other structures: ASCE standard 7-10*, American Society of Civil Engineers.

Abdollahzadeh, G., Kuchakzadeh, H. and Mirzagoltabar, A. (2017). "Performance-based plastic design of Moment Frame-Steel Plate Shear Wall as a dual system", *Civil Engineering Infrastructures Journal*, 50(1), 21-34.

Agency, F.E.M. (2009). *Quantification of building seismic performance factors*, FEMA P695, Washington, DC.

Akiyama, H. (1985). *Earthquake-resistant limit-state design for buildings*, University of Tokyo Press.

Calugaru, V. and Panagiotou, M. (2012). "Response of tall cantilever wall buildings to strong pulse type seismic excitation", *Earthquake Engineering and Structural Dynamics*, 41(9), 1301-1318.

Chao, S.-H., Goel, S.C. and Lee, S.-S. (2007). "A seismic design lateral force distribution based on inelastic state of structures", *Earthquake Spectra*, 23(3), 547-569.

Chinmayi, H. and Jayalekshmi, B. (2013). "Soil-structure interaction analysis of RC frame shear wall buildings over raft foundations under seismic loading", *International Journal of Scientific and Engineering Research*, 4(5), 99-102.

Choi, S.W., Kim, Y., Lee, J., Hong, K. and Park, H.S. (2013). "Minimum column-to-beam strength ratios for beam–hinge mechanisms based on multi-objective seismic design", *Journal of Constructional Steel Research*, 88, 53-62.

Fischinger, M. (2014). *Performance-based seismic engineering: Vision for an earthquake resilient society*, Springer

Goel, S.C. and Chao, S.-H. (2008). *Performance-based plastic design: earthquake-resistant steel structures*, International Code Council, Country Club Hills, IL.

Housner, G.W. (1956). "Limit design of structures to resist earthquakes", *Proceeding of the 1st WCEE*.

Housner, G.W. (1959). "Behavior of structures during earthquakes", *Journal of the Engineering Mechanics Division*, 85(4), 109-130.

Kato, B. and Akiyama, H. (1982). "Seismic design of steel buildings", *Journal of the Structural Division*, 108(8), 1709-1721.

Li, M., Lu, X., Lu, X. and Ye, L. (2014). "Influence of soil–structure interaction on seismic collapse resistance of super-tall buildings", *Journal of Rock Mechanics and Geotechnical Engineering*, 6(5), 477-485.

Liao, W.-C. (2010). *Performance-based plastic design of earthquake resistant reinforced concrete moment frames*, ProQuest, UMI Dissertations Publishing

Morshedifard, A. and Eskandari-Ghadi, M. (2017). "Coupled BE-FE scheme for three-dimensional dynamic interaction of a transversely isotropic half-space with a flexible structure", *Civil Engineering Infrastructures Journal*, 50(1), 95-118.

Mylonakis, G. and Gazetas, G. (2000). "Seismic soil-structure interaction: Beneficial or detrimental?", *Journal of Earthquake Engineering*, 4(03), 277-301.

Mylonakis, G., Syngros, C., Gazetas, G. and Tazoh, T. (2006). "The role of soil in the collapse of 18 piers of Hanshin Expressway in the Kobe earthquake", *Earthquake Engineering and Structural Dynamics*, 35(5), 547-575.

Poulos, H.G. and Davis, E.H. (1980). *Pile foundation analysis and design*, J. Wiley, New York.

Priestley, M., Calvi, G. and Kowalsky, M. (2007). "Direct displacement-based seismic design of structures", *5th New Zealand Society for Earthquake Engineering Conference*.

Riga, E., Karatzetzou, A., Mara, A. and Pitilakis, K. (2017). "Studying the uncertainties in the seismic risk assessment at urban scale applying the capacity spectrum method: The case of Thessaloniki", *Soil Dynamics and Earthquake Engineering*, 92, 9-24.

Sahoo, D.R. and Chao, S.-H. (2010). "Performance-based plastic design method for buckling-restrained braced frames", *Engineering Structures*, 32(9), 2950-2958.

Tabatabaiefar, S.H.R., Fatahi, B. and Samali, B. (2014). "An empirical relationship to determine lateral seismic response of mid-rise building frames under influence of soil–structure interaction", *The Structural Design of Tall and Special Buildings*, 23(7), 526-548.

Thermou, G., Elnashai, A. and Pantazopoulou, S. (2012). "Retrofit yield spectra, a practical device in seismic rehabilitation", *Earthquake and Structures*, 3(2), 141-168.

Verde, R.V. (1991). "Explanation for the numerous upper floor collapses during the 1985 Mexico City earthquake", *Earthquake Engineering and Structural Dynamics*, 20(3), 223-241.

Wongpakdee, N. and Leelataviwat, S. (2014). "Effects of column capacity on the seismic behavior of mid-rise strong-column-weak-beam moment frames", *Proceedings of the 2nd European Conference on Earthquake Engineering Seismology*, European Association for Earthquake Engineering, Istanbul, Turkey.

Wongpakdee, N. and Leelataviwat, S. (2017). "Influence of column strength and stiffness on the inelastic behavior of strong-column-weak-beam frames", *Journal of Structural Engineering*, 143(9), 04017124.