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Seismic performance of reinforced concrete shear wall buildings with underground stories

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Abstract. This paper investigates the seismic behavior of reinforced concrete shear wall buildings with multiple underground stories. A base-case where the buildings are modeled with a fixed condition at ground level is adopted, and then the number of basements is incrementally increased to evaluate changes in performance. Two subsurface site conditions, corresponding to very dense sands and medium dense sands, are used for the analysis. In addition, three ground shaking levels are used in the study. Results of the study indicated that while the common design practice of cropping the structure at the ground surface leads to conservative estimation of the base shear for taller and less rigid structures; it results in unpredicted and non-conservative trends for shorter and stiffer structures.

Keywords: soil structure interaction; shear wall buildings; underground stories; nonlinear time history analysis

1. Introduction

A controversial issue in the seismic analysis and design of buildings with multiple underground stories lies in incorporating the effects of the underground stories on the seismic response of these structures. Building codes lack recommendations concerning this controversy; thus, designers are currently basing their analyses on approximations, engineering judgment and experience. Some model and analyze the building cropped at the ground floor level, others include a certain number of basement floors, while few include all the underground floors. Explicitly incorporating the underground stories and the associated soil in the mathematical model of the structure will allow the designer to accurately assess the building's seismic performance.

For buildings with underground stories or basement walls, soil structure interaction (SSI) can occur at two different levels: at the foundation level and at the interface between the basement walls and the side soil. Soil structure interaction has been an active area of research over the past decades due its controversial outcomes and its engineering importance. Incorporating the nonlinear hysteretic behavior of the SSI may lead to increased energy dissipation and effective damping leading to changes in the force demand on the structure. Moreover, modeling the foundation and

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side soil flexibility can alter the fundamental period of the system and consequently affect the structural response of the building.

A review of the literature indicates that there are few studies that are aimed at investigating the effect of basement walls on the seismic response of structures while including SSI concepts (ex. El-Ganainy and El Naggar 2009, Maleki and Mahjoubi 2010). On the other hand, studies that investigated the effects of foundation flexibility on the seismic response of structures are numerous and show a systematic improvement in the approaches and methodologies adopted in modeling the soil structure interaction. A description of the methodologies used in these studies to model the structure and the soil is presented to illustrate the development that is evident in modeling the different aspects of soil structure interaction.

1.1 Soil-structure interaction at foundation/soil interface

Chopra and Yim (1985) compared the seismic performance of single degree of freedom structures that are supported on rigid foundation soil and flexible foundation soil, with the soil flexibility being reflected by two spring-damper elements, Winkler foundation with distributed spring-damper elements or a visco-elastic half-space. Mylonakis and Gazetas (2000) used elastic and elasto-plastic single degree of freedom oscillators to evaluate the approach that seismic regulations propose for assessing SSI effects. A two-degree-of-freedom system was adopted to investigate the role of SSI on the inelastic performance of a bridge pier. Ambrosini *et al*. (2000) studied the effect of foundation flexibility on the seismic response (base shear and moments) of structures with prismatic rectangular foundations by assuming that both the soil and the structure are linearly elastic. Dutta *et al*. (2004) studied the same effect on 3-dimensional low rise building frames resting on shallow foundations while assuming that the structure is inelastic and that the soil is clay that could be modeled by a set of six elastic translational and rotational springs.

Raychowdhury (2009, 2011) used the nonlinear Winkler springs approach to simulate the response of low-rise steel buildings supported by isolated foundations on dense silty sand under earthquake loading. The obtained results show that the effect of soil nonlinearity may be significant when a building is supported by shallow foundations whose capacity may be mobilized under strong earthquake shaking. Tabatabaiefar and Massumi (2010) used the finite element approach to study the effects of soil structure interaction on reinforced concrete moment resisting frames. In the analysis, structures consisting of 3, 5, 7, and 10 stories were modeled using frame elements while 3-dimensional quadrilateral elements were used for modeling the soil. The soil was modeled as elastic with a secant shear modulus and internal soil damping that are consistent with a shear strain magnitude of 0.1.

Moghaddasi *et al*. (2011, 2012) used Monte Carlo simulations to investigate the impact of uncertainties in the soil-structure interaction parameters on the response of a linear single degree of freedom superstructure that is supported by an equivalent linear soil half space. The equivalent linear soil model represents the soil nonlinearity by using a reduced soil modulus and an increased damping in accordance with the strain level encountered. Tang and Zhang (2011) investigated the effect of soil structure interaction on the response of mid-rise (7-storey) slender reinforced concrete shear wall for a number of earthquake ground motions in a probabilistic framework. In the study, both the linear foundation impedance model (linear springs and dashpots) and the beam-on-nonlinear-Winkler foundation approach were utilized to model the footing and the soil supporting the wall. Renzi *et al*. (2013) used idealized single degree of freedom systems that represent ordinary shear-type buildings (up to 20 storeys) with surface square rigid foundations that were modeled using impedance functions that were obtained using the finite element method or a homogeneous viscoelastic half-space. Jarernprasert *et al*. (2013) examined the response of single-storey inelastic structures with foundations embedded in an elastic half-space under seismic input that was obtained from earthquake records from California and Mexico City.

Anastasopoulos and Kontoroupi (2014) conducted a 3-dimensional finite element analysis that models the entire soil-foundation-structure system while taking into consideration material and geometrical nonlinearities. The FE analysis was used to formulate a simplified model for the seismic response that is based on a single degree of freedom system that is supported by a nonlinear rotational spring, a linear rotational dashpot, and linear horizontal and vertical springs and dashpots. Torabi and Rayhani (2014) also developed a 3-dimensional dynamic Finite Element model that captures the seismic response of the soil-foundation-structure system for foundations on soft saturated clay. The analysis was performed for linearly elastic structures represented by a single degree of freedom system supported by elastic foundation on inelastic clay that was modeled with an elasto-plastic constitutive model. Tabatabaiefar and Fathi (2014) used the finite difference software FLAC 2D to determine the inelastic seismic response of mid-rise (5, 10, and 15 storey) moment resisting concrete building frames that are connected to plain strain foundation soil elements using frictional interface elements. The Mohr-Coulomb constitutive model was used to model the soil response under seismic loading.

Common findings from the literature on the soil structure interaction at the foundation/soil level are summarized below:

1. A reduction in the structural load demands was observed when the foundation SSI was incorporated, particularly for structures with fundamental periods that were close to the peak or on the descending branch of the response spectra adopted (Chopra and Yim 1985, Ambrosini *et al*. 2000, Tabatabaiefar and Massumi 2010, Jarernprasert *et al*. 2013).

2. The general belief of the reduction in load demands due to SSI is not always valid. This is particularly true for low rise structures with small natural periods which showed increases in the seismic base shear when SSI is incorporated due to the fact that they lie in the sharply increasing zone of the response spectrum (Mylonakis and Gazetas 2000, Dutta *et al*. 2004, Moghaddasi *et al*. 2011, Moghaddasi *et al*. 2012, Jarernprasert *et al*. 2013).

3. The significance of incorporating SSI is related to the soil type and properties, the number of stories and/or the combination of both. Increases in seismic base shears in low rise buildings due to soil flexibility seems to decrease with increasing hardness of soil and increasing number of stories. On the other hand, rigid tall structures can undergo significant natural period elongation that is accompanied with significant foundation rocking particularly for softer soils (Dutta *et al*. 2004, Tabatabaiefar and Massumi 2010, Tang and Zhang 2011, Torabi and Rayhani 2014, Tabatabaiefar and Fatahi 2014).

4. When linear SSI models are considered, the structural demands are expected to be higher than the corresponding nonlinear SSI indicating that the linear SSI models will likely lead to more conservative designs (Raychowdhury 2009, Raychowdhury 2011).

5. The inelastic range demands (particularly displacements) of lateral load resisting elements may increase due to the effect of soil flexibility compared to the fixed base case (Dutta *et al*. 2004, Tabatabaiefar and Massumi 2010, Moghaddasi *et al*. 2011, Moghaddasi *et al*. 2012).

As evidenced from the above studies, the procedures used for modeling the soil-foundation-structure system in SSI problems range from the more complex (modeling the soil as a continuum using finite element or finite difference packages) to the simple and practical (effective linear spring models). Dutta and Roy (2002) presented a comparative review of the commonly used soil modeling techniques. With regards to the foundation model, the authors recommended that the use of nonlinear soil models to represent shallow foundations will yield optimal results. However the study also noted that the linear Winkler hypothesis, despite its obvious limitations, yields reasonable performance and is relatively simple to implement.

1.2 Soil-structure interaction at wall/soil interface

For buildings with underground stories, the soil-structure-interaction between the side soil and the basement walls affects the response of the structure under seismic excitation. Several empirical and numerical methods have been used for modeling the effects of side SSI. Although these methods are based on slightly varying assumptions, they can generally be classified into three main groups: the simple lateral earth pressure methods, the beam column methods, and the finite element methods. The traditional lateral earth pressure methods replace the soil by a seismic earth pressure distribution that could be evaluated by the Mononobe-Okabe method (Okabe 1926, Mononobe and Matsuo 1929). The method is a pseudo-static method that is based on force equilibrium and Coulomb's wedge theory for the active and passive earth pressures. Ostadan (2005) reports that the Mononobe-Okabe method is applicable for retaining walls and that field observations and experimental data show that the assumptions used in the method are not applicable to building walls.

Under the category of studies that utilize the beam column methods for modeling the lateral seismic earth pressure are the studies of Veletsos and Younan (1994) and Richards *et al*. (1999). Veletsos and Younan (1994) proposed a method for calculating the dynamic soil pressures on rigid vertical walls by modeling the soil medium by a series of elastically supported, semi-infinite horizontal bars with distributed mass (rather than by springs of constant stiffness) and impedances that depend on the ratio of the exciting frequency to the natural frequency of the soil medium and on the material damping factor of the soil. Richards *et al*. (1999) developed a simple kinematic method to determine the distribution of dynamic earth pressure on retaining structures. The soil in this method which takes into account the plastic non-linear response in the free field is modeled by a series of springs with stiffnesses that are related to the soil's elastic or secant shear modulus. Ostadan (2005) presented a simplified method for calculating the lateral seismic earth pressure on below-ground building walls that are resting on firm foundations (fixed base). The method which makes use of a single degree of freedom system to model the structure incorporates the dynamic soil properties and the frequency content of design motion in its formulation.

El-Ganainy and El-Naggar (2009) conducted the first attempt to model the seismic performance of 3-dimensional moment resisting steel frame buildings with multiple underground stories. In the nonlinear structural analysis that was conducted, the soil under the foundations and next to the basement walls was assumed to experience nonlinear behaviour under seismic shaking through the beam on a nonlinear Winkler springs approach. In the analysis, the radiation damping through the foundation soil and the side soil was neglected. It was also assumed that the dynamic lateral earth pressure along the basement walls is bounded by the active and passive earth pressure, respectively, and that the relationship between the lateral earth pressure and the wall displacement could be modeled using the concept of p-y curves as proposed by Briaud and Kim (1998). Briaud and Kim (1998) recommended specific p-y curves for walls in sand based on an analysis of the results of full scale experimental tests of tie-back walls in sand. Maleki and Mahjoubi (2010) presented solutions for the seismic earth pressure against retaining walls with different boundary and stiffness conditions ranging from rigid walls to flexible walls. Non-linear finite element dynamic time history analyses of soil-wall systems were employed for verification of the superiority of the proposed solutions to the well-known Mononobe-Okabe relationships. The main limitation of the proposed solution is that it is based on the assumption that the base of the wall is fixed, which might not be the case for walls supported on shallow foundations.

1.3 Objective of study

Except for the study conducted in El-Ganainy and El-Naggar (2009), none of the proposed methods for evaluating the lateral earth pressure on basement walls incorporates the complete model of the superstructure in their formulation. In addition, the recommended earth pressure distributions from these methods do not explicitly include the effect of soil structure interaction due to the flexibility of the foundations/soil interface. In other words, the recommended lateral earth pressures under seismic shaking in these studies could be considered to be decoupled from the response of the foundation of the structure and the basement walls. In addition, the results presented in El-Ganainy and El-Naggar (2009) are applicable to moment resisting steel frame buildings with multiple underground stories and may not be applicable for more rigid shear-wall supported reinforced concrete buildings with underground stories. There is a need for investigating the seismic performance of reinforced concrete shear wall buildings with underground stories using a model that would incorporate the superstructure, the soil-structure interaction at the foundation level, and the soil structure interaction between the basement walls and the side soil.

The main objective of this study is to investigate the combined effects of side and foundation soil structure interactions for reinforced concrete shear wall buildings under seismic loading. The impacts of the building substructure on its seismic performance are gauged by explicitly incorporating the underground stories, basement walls, foundations and side soil in the structural analysis model. Two subsurface site conditions corresponding to very dense sands and medium dense sands are used in the study. Moreover, the system is loaded via three different levels of ground shaking. Nonlinear direct integration time history analysis of the reinforced concrete shear wall building is performed using SAP2000 (Computers and Structures Inc., 2007). A sensitivity analysis with the following varying parameters is conducted:

1. Number of above ground stories
2. Number of underground stories
3. Subsurface soil conditions

For each scenario, the base shear and the inter-storey shear demands are evaluated in order to quantify the effects of soil structure interaction on the design process.

2. Analysis approach

The parametric sensitivity analysis involves evaluating the seismic response of different buildings while varying the number of above ground floors, underground floors, and site conditions. The building sites are assumed to have a deep homogeneous soil deposit underlain by bedrock. Two soil classes are adopted for modeling the soil in this study: soil class C corresponding to “very dense soil or soft rock” and soil class D corresponding to “stiff soil”, in accordance with ASCE 7-10 (American Society of Civil Engineers, 2010). In both classes, the soil is assumed to be comprised of granular material. The soil structure interaction effects are modeled using the multi-linear kinematic plastic link property of SAP2000 (Computers and Structures Inc., 2007). A base-case where the buildings are modeled with a fixed condition at ground level is adopted, and then the number of basements is incrementally increased to investigate changes in performance.

2.1 Building model

The structures considered in this study are typical reinforced concrete shear wall buildings. Fig. 1 shows a typical plan of the buildings considered (Ghosh and Khunita 1999). The slab is approximately 550 m^2 with 6 transverse 5 m spans and 3 longitudinal 6 m spans. A constant floor height of 3 m is assumed for all buildings. The preliminary design of the five, ten, and fifteen storey buildings is carried out using the structural analysis program ETABS ((Computers and Structures Inc., 2007) assuming fixed base conditions at the ground surface. The basement walls are designed to resist bearing and lateral earth pressure loads only. The slab is designed as a 20 cm thick post-tensioned flat slab. All buildings are designed to be resting on spread and strip footings. The gravity loads assigned to the buildings are the own weights of the structural components including the reinforced concrete beams, columns, slabs and basement and shear walls. The weights of the non-structural components (e.g., cladding, tiling, partitions, finishing, etc.) are modeled as a superimposed uniform load equal to 4 kN/m^2. A uniformly distributed live load of 2 kN/m^2 is used for all residential areas and 3 kN/m^2 for parking zones as per the ASCE 7-10 load requirement criteria. The shear walls are 20 cm thick and are designed to carry the entire lateral load. Although the frame of the building is integrated with the shear wall system, the frame-wall interaction is assumed to provide an extra safety factor when it comes to lateral load resistance. Members of the frame (columns, beams and slabs) are proportioned to resist gravity loads only. Once the sections are sized based on the ETABS model output, replicate models are developed in SAP2000 to allow for incorporating the SSI effects. Fig. 2 shows three typical two dimensional sections of the five storey models corresponding to the fixed base, flexible base, and three-basement scenarios.

2.2 Soil Model

The analysis conducted in this study involves soil-structure interaction at the foundation level and along the basement walls. Since the main focus is the effect of basement walls on the seismic

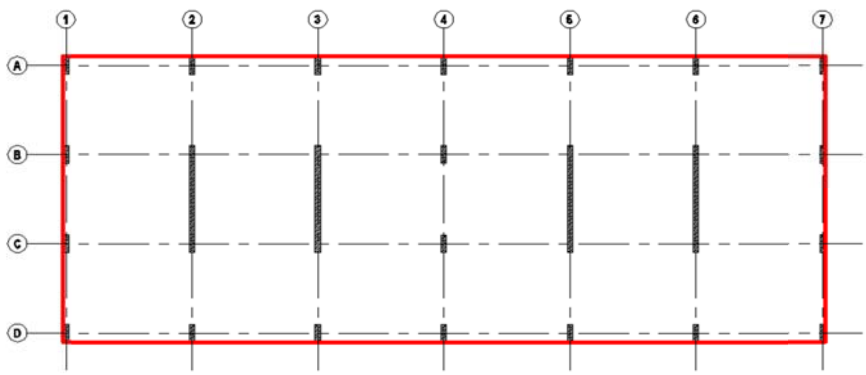


Fig. 1 Typical building floor plan

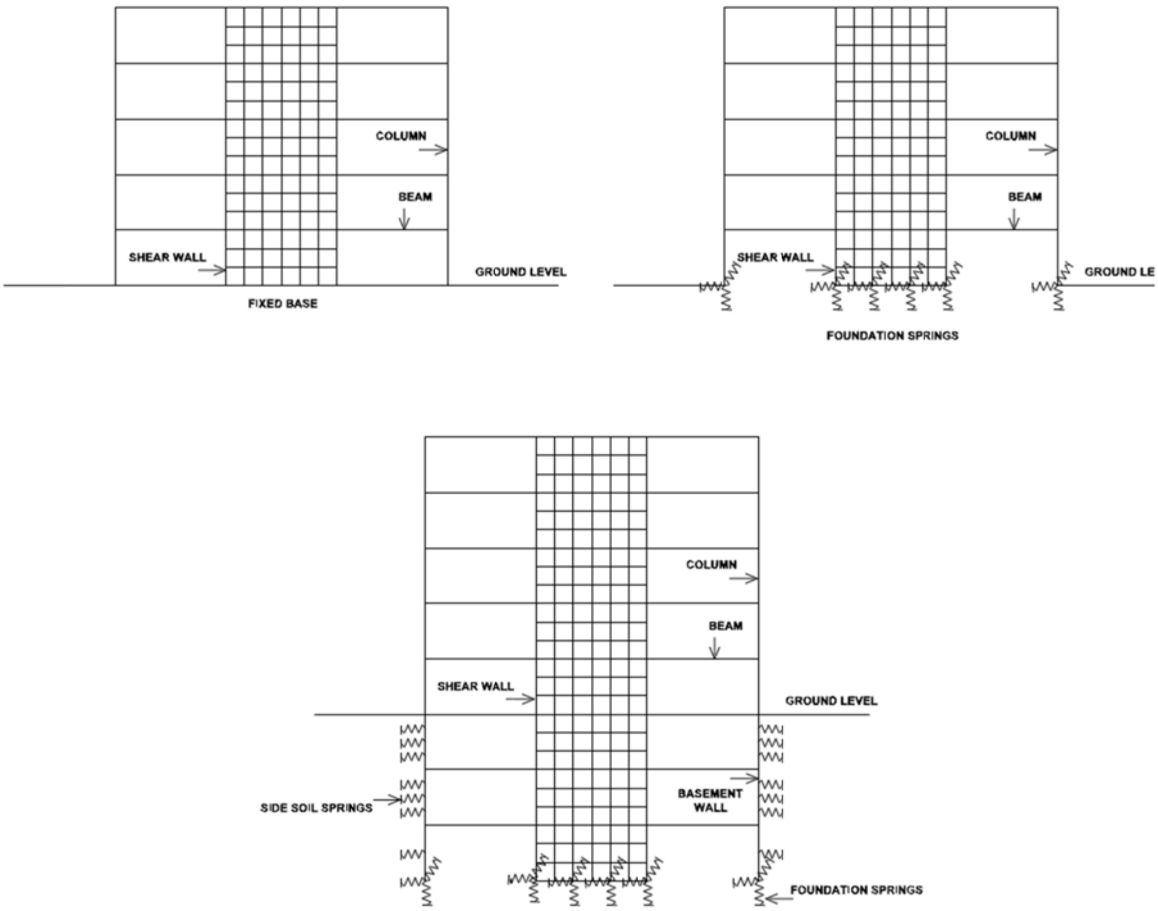


Fig. 2 The mathematical models used for the analyses of the different building models; fixed base case (upper left), flexible base (upper right), and 3 basements (lower middle)

response of the reinforced concrete structure, a decision was made to adopt an established and simple approach for modeling the nonlinear foundation-soil interaction in lieu of more sophisticated approaches with large number of parameters and assumptions. The foundation system of the buildings is comprised of a network of shallow spread footings. Two types of shallow footings are identified: one for interior columns and another for edge columns. The foundations of the basement and shear walls are designed as strip footings. Shallow and strip footings are designed based on Meyerhof's bearing capacity theory and the elastic settlement theory Das (Das 2007), and the design is then checked for one-way and two-way shear failure according to ACI 318-14 (American Concrete Institute, 2014). Table 1 presents the governing soil parameters for the different site classes used in the study. The soil parameters are estimated using common empirical correlations with the ASCE site classification system.

The vertical, horizontal, and rotational elastic stiffnesses of the footings are calculated using the frequency independent formulas as presented in the ASCE 41-13 report (American Society of Civil Engineers, 2013). The stiffnesses of vertical and horizontal springs (translation) and that of

Table 1 Soil model parameters

	Soil Class C	Soil Class D
Angle of internal friction, ϕ’	42	37
Dry unit weight, $\gamma_{\mathrm{dry}}(\mathrm{kN/m^3})$	20	19
Poisson’s ratio, ν	0.4	0.3
Shear wave velocity, V_s (m/s)	500	275
Material damping ratio	0.05	0.05
Relative density, (%)	90	65
Initial shear modulus, $\mathrm{G_0} = \frac{\gamma V_s^2}{g}$ (kPA)	510,000	146,500

the rotational spring (rocking) are calculated as a function of the footing dimensions and the initial shear modulus of the soil and assigned to the model node of the respective footing. The calculated spring constants are then corrected for the effect of footing embedment as per ASCE 41-13 recommendations. The embedment correction factor is determined as a function of the footing embedment depth and the footing width, length, and thickness.

To account for the degradation that occurs in the stiffness of the springs due to the unload-reload cycles that are associated with seismic shaking, the initial elastic stiffnesses of the springs were replaced with reduced equivalent nonlinear stiffnesses (referred to as effective stiffnesses in the literature). As a result, an effective shear modulus that is a function of the soil class was calculated as per the recommendations of ASCE 41-13, whereby effective shear modulus ratios G/G_0 are proposed as a function of the site class and the effective peak acceleration of the selected earthquake. For the range of levels of shaking that are anticipated in this study and in line with the ASCE 41-13 recommendations, the effective shear moduli were evaluated by multiplying the initial elastic shear modulus G_0 by reduction factors of 0.85 and 0.75 for soil classes C and D, respectively. These modulus reduction factors are in line with reduction factors that were adopted by other researchers for similar seismic and soil conditions (El Ganainy and El Naggar 2009).

The side soil next to the basement walls was modeled using non-linear springs described by a hysteretic lateral pressure versus lateral displacement relationship that is consistent with the p-y curves concept commonly used to model the reaction of the soil for laterally loaded piles. Briaud and Kim (1998) were the first to recommend p-y relationships for the analysis and design of tie-back walls. These relationships were calibrated/back calculated using data collected from full scale tests on walls in sand. Briaud and Kim (1998) state that the lateral earth pressure that is exerted by the soil on the wall is bounded by the active and passive earth pressure conditions. Based on the data collected, they recommend that the active earth pressure P_a and the passive earth pressure P_p could be assumed to be mobilized at wall movements of 1.3 mm (away from the retained soil) and 13 mm (into the retained soil), respectively. El Ganainy and El Naggar (2009) adopted this p-y relationship in their analysis of the seismic response of moment resisting frames with underground stories. Fig. 3(a) presents the backbone curve of the lateral pressure-lateral deflection curve used for modeling the side soil. The earth pressures at a given depth are dependent on the soil type and properties and on the embedment depth and are given by

$$P_a = K_a . \gamma . z . \cos \delta \tag{1}$$

$$P_p = K_p.\gamma.z.\cos\delta \tag{2}$$

where,

$$K_a = \frac{\cos^2\phi}{\cos\delta\left[1+\frac{\sqrt{\sin(\delta+\phi)\sin\phi}}{\cos\delta}\right]^2} \tag{3}$$

$$K_p = \frac{\cos^2\phi}{\cos\delta\left[1-\frac{\sqrt{\sin(\delta+\phi)\sin\phi}}{\cos\delta}\right]^2} \tag{4}$$

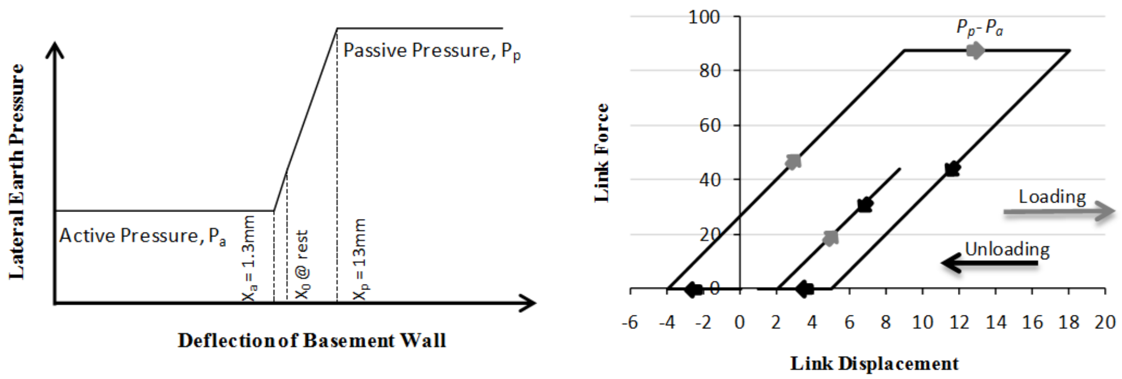
and γ is the unit weight of soil, z is the embedment depth at which the soil pressure is calculated, δ is the wall-soil friction angle, and ϕ is the angle of friction of the soil.

The multi-linear plastic kinematic link property in SAP2000 is used to model the p-y curves. One of the limitations of this link is the fact that it requires the load-deformation relationship to pass by (0,0) which renders P_p and P_a with opposite signs and falsely indicates that one is in tension while the other is in compression. To mitigate this shortcoming, the p-y curves were modeled by the superposition of two components as recommended by El Ganainy and El Naggar (2009):

• A bi-linear link bounded by a maximum of (P_p-P_a) and a minimum of 0. The SAP2000 plastic link property is used to model this behaviour.

• A constant active pressure P_a applied to the basement walls. This is applied as a static lateral load on the basement walls of the building.

To illustrate the force-displacement behavior of the links, a segment of a response of a link is presented in Fig. 3(b). The grey arrows represent the loading phase (wall moves towards the soil) while the black arrows represent the unloading phase (wall moves away from the soil). Before the cyclic loading is initiated, active conditions are assumed to act on the wall forcing the cycle to



(a) (b)

Fig. 3 (a) Backbone curve of the lateral pressure-lateral deflection model of the side soil, and (b) example of hysteretic link/spring load-displacement relation initiate from position (0, 0). First, the wall moves away from the soil (curve moves to the left) for a distance of 4 mm resulting in no additional stresses to the already applied active pressure P_a. Then, the wall starts moving into the soil along a loading (compression) path until the force in the link hits the maximum passive reaction of P_p-P_a. Once this plateau is reached, the reaction remains constant until an unloading cycle begins. The loading-unloading process travels along a line having a constant slope (average equivalent stiffness of the spring). Once the force in the spring/link reaches zero (active conditions), it remains zero until another loading cycle starts. To avoid computational complexities, this study neglects the effects of the oscillating mass of the side soil, radiation damping, and soil gapping. El-Ganainy and El-Naggar (2009) presents a comprehensive discussion in relation to the impact of these factors on the seismic response of structures with underground stories.

In this study, the static earth pressure parameters, K_a *and* K_p, are used in lieu of the their seismic counterparts, K_{ae} *and* K_{pe}, since the latter parameters are primarily used for pseudo-static analyses that adopt the maximum horizontal acceleration as a basis for design. This study involves nonlinear direct integration time history analyses to evaluate the seismic response of the building models. In such analyses, the ground acceleration record varies with time and the lateral earth pressures that are mobilized in the different springs are fully coupled with the associated acceleration record.

Moreover, K_{ae} *and* K_{pe} are generally used to estimate the total thrust (resultant) of lateral earth pressure. There is controversy in the literature on the point of application of the total thrust and on the variation of the seismic lateral earth pressure along the depth of the wall (Sitar *et al.* 2012). Since the side soil model adopted in this study considers the variation of the soil springs that are distributed over the height of the wall, the total seismic thrust estimated using K_{ae} *and* K_{pe} may not be applicable for modeling the hysteretic backbone p-y relationship for the side soil.

2.3 Earthquake loads

Three representative earthquake loads are chosen to cover a relatively wide range of design response spectra. Ground acceleration time histories consistent with the 1940 El-Centro Earthquake, 1987 Pasadena Earthquake, and 1990 Pomona Earthquake are used in this study. The earthquake records for the different ground motions are shown in Fig. 4. Table 2 summarizes the main characteristics of the ground motions used (Pacific Earthquake Engineering Research Center 2005) and indicates that the El-Centro ground motion has the longest duration (26 seconds between the first and last exceedance of 0.05 g), followed by the Pasadena shaking (13.5 seconds) and the Pomona shaking (4 seconds). The peak ground acceleration (PGA) of the El-Centro motion is also the highest (around 0.31 g) followed by the Pomona and Pasadena earthquakes with PGAs of 0.22 g and 0.20 g, respectively. In order to highlight the frequency content of the different ground excitations, the response spectra, assuming 5% damping, of the three earthquakes are shown in Fig. 5. Each building is analyzed for each of these three records.

2.4 Nonlinear time history analysis

Nonlinear direct integration time history analyses are performed to evaluate the seismic response of the building models. While nonlinear springs are used to simulate the soil response,

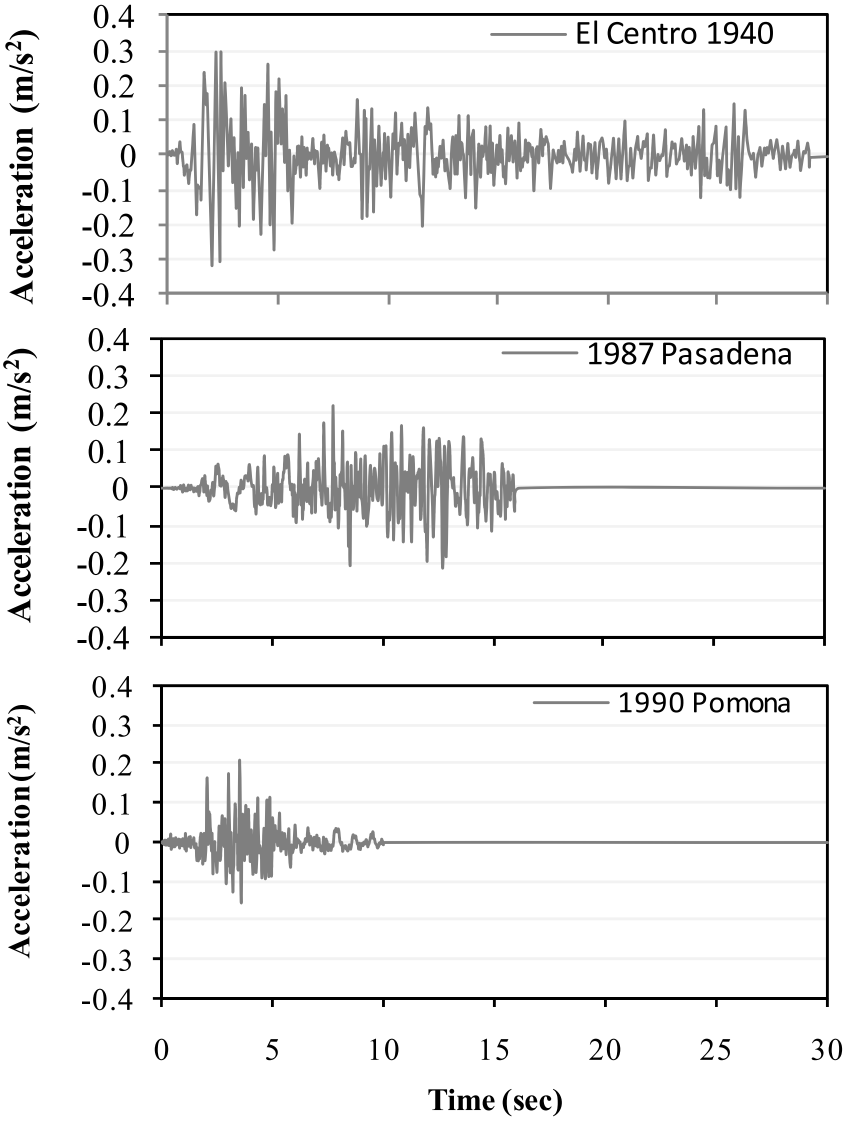


Fig. 4 (1) 1940 El Centro ground motion, (2) 1987 Pasadena ground motion, and (3) 1990 Pomona ground motion

Table 2 Summary of the different ground motion characteristics

	1940 El-Centro	**1987 Pasadena**	**1990 Pomona**
Peak ground accel., PGA (g)	0.31	0.22	0.207
Sustained max. accel. 3 cycles (g)	0.298	0.208	0.164
Sustained max. accel. 5 cycles (g)	0.272	0.185	0.125
Duration (sec)	25.98	13.52	3.96
Richter Magnitude	6.9	5.9	4.8
Mercalli Magnitude	X	VII	V

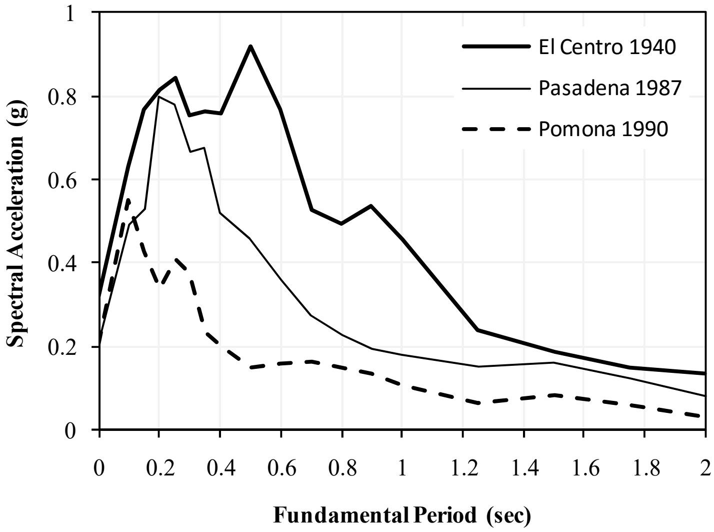


Fig. 5 The Response spectra of the three ground motions, ($\zeta = 5\%$)

the structure is assumed to be perfectly elastic. Proportional damping is adopted to simulate energy dissipation resulting from local structural hinging or plastic action. Proportional damping, also known as Rayleigh damping, defines the global damping matrix as a linear combination of the global mass and stiffness matrices. This renders the structural damping frequency dependent. Consequently, for each building model, the frequency range of interest is identified, and the Rayleigh damping parameters are automatically calculated by SAP2000 to consider 5% of critical damping in each mode of vibration.

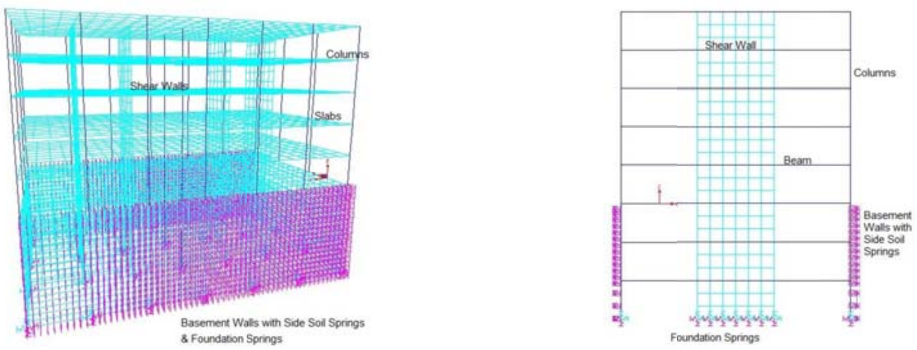
The analysis is conducted using the nonlinear structural analysis software SAP2000 (Computers and Structures Inc. 2007). The Hilber-Hughes-Taylor (HHT) time integration method is used for the analysis. The HHT is a robust time integration technique that requires one input parameter α which can take values between 0 and -1/3. In the analyses models, $\alpha = 0$ is used; this renders the HHT method equivalent to the Newmark method with $\gamma = 0.5$ and $\beta = 0.25$ (Computers and Structures Inc. 2007). The ground motions are given in 0.02 sec time increments. Using an adaptive time integration refinement approach, a time step of 0.005 seconds is selected for the analyses. The input ground motions are linearly interpolated to obtain the intermediary values. The initial conditions for the dynamic analyses cases are selected as the deformed structural configuration resulting from the application of the dead and superimposed dead loads. For each building model, the response is studied with and without incorporating the soil-flexibility effects. Variations in the base shear and inter-storey shear demands are explained due to the changes of various governing parameters.

3. Results and discussion

The analyzed models are classified into three categories depending on the number of above ground floors in each model: five stories, ten stories or fifteen stories. Each category is first analyzed following the common practice of cropping the structure at ground level and applying fixed support conditions at that level i.e., ignoring the effects of SSI. Then the structure is analyzed assuming no basement walls but taking into effect a flexible foundation/soil system. Ultimately, the basements are included into the building model to incorporate the effects of both foundation and side soil structure interactions on the response. For buildings with 5 above-ground stories, the analysis was conducted for sites with soils that categorized as Soil Class C and Soil Class D. For the 10 and 15 storey buildings, only cases with Soil Class C were analyzed. This is attributed to the fact that the analysis of the 10 and 15 storey buildings under static loading conditions indicated the need for a raft foundation option or a deep foundation option to support the structures and limit settlements to acceptable values. The proper modeling of these foundation systems with the structure is beyond the scope of this paper. As a result, the analyses for 10 and 15 storey buildings were limited to sites with soil class C where spread footings could be used to support the load.

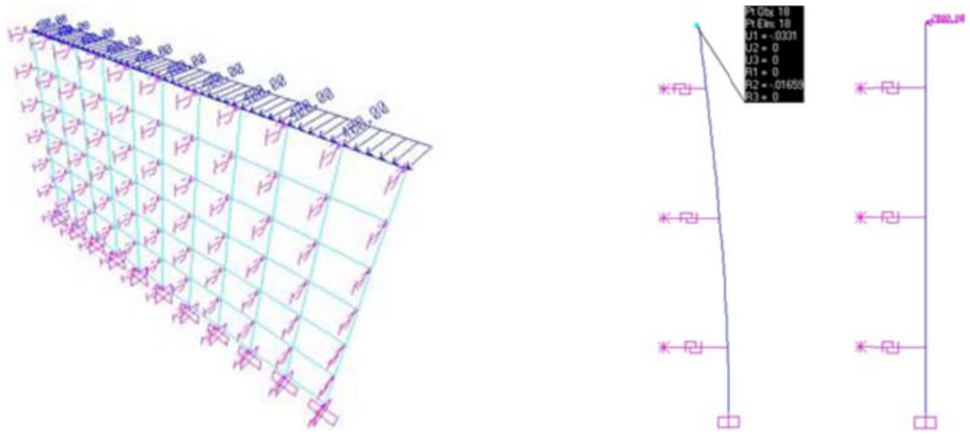
All buildings were analyzed for each of the three ground motions presented earlier. It is important to note that initially a 3D model of the 5 story building was developed using SAP2000. But since the time history simulations were computationally demanding and time consuming, the authors reverted to using 2D models for the analysis. Fig. 6 below shows snapshots of the 3D and 2D models simulating the behavior of the 5 story building with 3 basement floors.

The 2D model simulates the behavior of the frame on gridline 2 (or gridline 3, 5 or 6) shown in Fig. 1. The columns and shear walls remain the same as the 3D model while the slab was replaced by an equivalent beam element having the same structural stiffness. A similar approach was adopted for the basement walls as they were also replaced with beam elements having an equivalent structural stiffness. In both the 2D and 3D models, the same side soil backbone curve was used to model the variation of the lateral earth pressure with lateral displacement. The only difference between the 2D and 3D models is the tributary area that is assigned to each spring. Fig. 7 shows a schematic of a simple test conducted to validate the 3D-2D conversion with regards to modeling the side walls and the side soil. In the 3D model of the wall, the springs are located in a grid at the node of each finite element. The forces in each spring at any given lateral displacement are obtained from the back-bone p-y curve by multiplying the reaction stresses with the spring tributary area (area of an element in the 3D case). In the 2D case, the springs are distributed vertically at a given spacing. In this case, the reaction forces in the spring are also obtained from the backbone curve through multiplying the reaction stresses by the tributary area, which is assumed in the 2D case to be equal to the vertical spacing of the springs multiplied by the width of a typical bay.



(a) (b)

Fig. 6 SAP2000 Analysis Models (a) 3D, and (b) 2D model of one bay



(a) (b)

Fig. 7 (a) 3D wall model with side springs and load, and (b) 2D wall Model with side springs and load

A comparison between the 2D and 3D cases indicated similarity in the wall response. In addition, a comparison between the base shears calculated for the case of the 5-storey 3D structure (with 3 basements) and the equivalent 2D simplified model of one bay indicated that the base shear of the bay (2D case) is almost 1/6 of the total base shear calculated in the full 3D structure (which includes 6 bays). This observation confirms the validity and applicability of the simplified approach adopted in this paper for studying the SSI effects using simplified 2D structures in lieu of the costly 3D case. This does not imply that the 2D model assumption could replace the 3D structure in other more complicated structures.

Table 3 presents the fundamental periods of the three building categories for the different soil profiles. As expected, the fundamental periods of the flexible base buildings were found to be larger than periods of the corresponding fixed base buildings, irrespective of the number of basements. In addition, the period for a given building was found to be higher for soil Class D compared to soil Class C and also higher for the buildings with the larger number of basement floors. The fundamental periods presented in Table 3 were utilized in association with the response

Table 3 Fundamental periods of the three building categories for the different site conditions

	5 Stories T (sec)	10 Stories T (sec)	15 Stories T (sec)
Fixed Base	0.25	0.75	1.37
Flexible - No Basements			
Soil Class C	0.35	0.88	1.53
Soil Class D	0.68	NA	NA
3 Basement Floors			
Soil Class C	0.40	0.94	1.60
Soil Class D	0.46	NA	NA
5 Basement Floors			
Soil Class C	NA	0.96	1.62
Soil Class D	NA	NA	NA

spectra of the different earthquakes as presented in Fig. 5 to quantify the changes in the spectral accelerations as a result of changing the condition of a building from the conventional fixed base scenario to the more realistic flexible base scenario in which the effects of the existence of basement floors and the soil's stiffness are expected to be significant. Changes in the spectral accelerations will in turn affect the response of the structure, particularly with regards to the magnitude of the inter-storey and base shear demands.

3.1 Effect of SSI on seismic response of 5-Storey buildings

To illustrate the effect of foundation flexibility and the presence of basement floors on the seismic response of the structure, the response of the 5-storey building was analyzed for the El-Centro ground motion for the case with Soil Class C (SC). The analysis was conducted for three cases: (1) fixed base case with no basement floors, (2) flexible base case with no basement floors, and (3) flexible base case with 3 basement floors. Fig. 8(a) shows the envelope of the storey shear demands for the three cases under consideration. Also shown on the figure are the calculated values of base shear (taken as the shear at ground level in this paper), peak spectral acceleration, and horizontal and vertical displacements computed at the ground level at the instant of peak load demand. The spectral accelerations and the displacements for the different scenarios are plotted to provide feedback on the factors affecting the computed storey/base shear demands and the overall building response under seismic shaking.

An analysis of the results on Fig. 8(a) leads to the following observations: (1) the storey shear was found to decrease in the flexible base case (no basements) compared to the base case where the building was assumed to be fixed at the foundation level. In particular, the decrease in the base shear demand was in the order of 25%, (2) when 3 basement floors with flexible foundations and side soil were included in the building models, the calculated storey shear was found to increase significantly, with increases of about 34% noted in the base shear in comparison to the fixed base case.

For the cases with zero basements, the drop in the storey shear demand due to the flexible foundations is expected given the drop that was witnessed in the peak spectral accelerations (see Fig. 8(a)) for the flexible case compared to the fixed base case. For the case with 3 basements, calculated peak spectral accelerations also decreased compared to the fixed base case. However, this decrease was accompanied by a significant increase in the base shear, indicating that factors other than the change in spectral acceleration played a role in increasing the base shear for the 3 basement case.

The increase in the base shear for the 3-basement case could only be attributed to the lateral earth pressures that are mobilized in the side soil as the basement walls displace laterally during seismic shaking. An analysis of the building movements at ground level in Fig. 8(a) indicates that the displacements for the 3-basement case are in the order of 18 mm. Displacements of this magnitude will mobilize significant lateral earth pressure from the side soil springs resulting in additional lateral forces that were non-existent in the cases with no basements. This is illustrated in Fig. 9(a) where the mobilized lateral forces in the springs along the 3 basements wall are presented along with an indication of the percent activation of the capacity of these springs. The percent activation of spring forces is defined as the ratio of the mobilized ultimate reaction of any given spring to the maximum nominal capacity of the spring (equivalent to the mobilization of passive pressure at the location of the spring). At the instant of peak load demand, the lateral displacement of the basement walls was sufficient to mobilize 100% passive reactions in the majority of the springs of the 1st basement wall where the lateral displacements were the largest, with the percent activation of lateral reactions decreasing at the levels of the 2nd and 3rd basement walls. The relatively high activation of spring reactions, coupled with the relatively high magnitude of the reactions for Soil Class C resulted in the significant increase in the base shear compared to the fixed and flexible base scenarios with no basements (Fig. 8(a)).

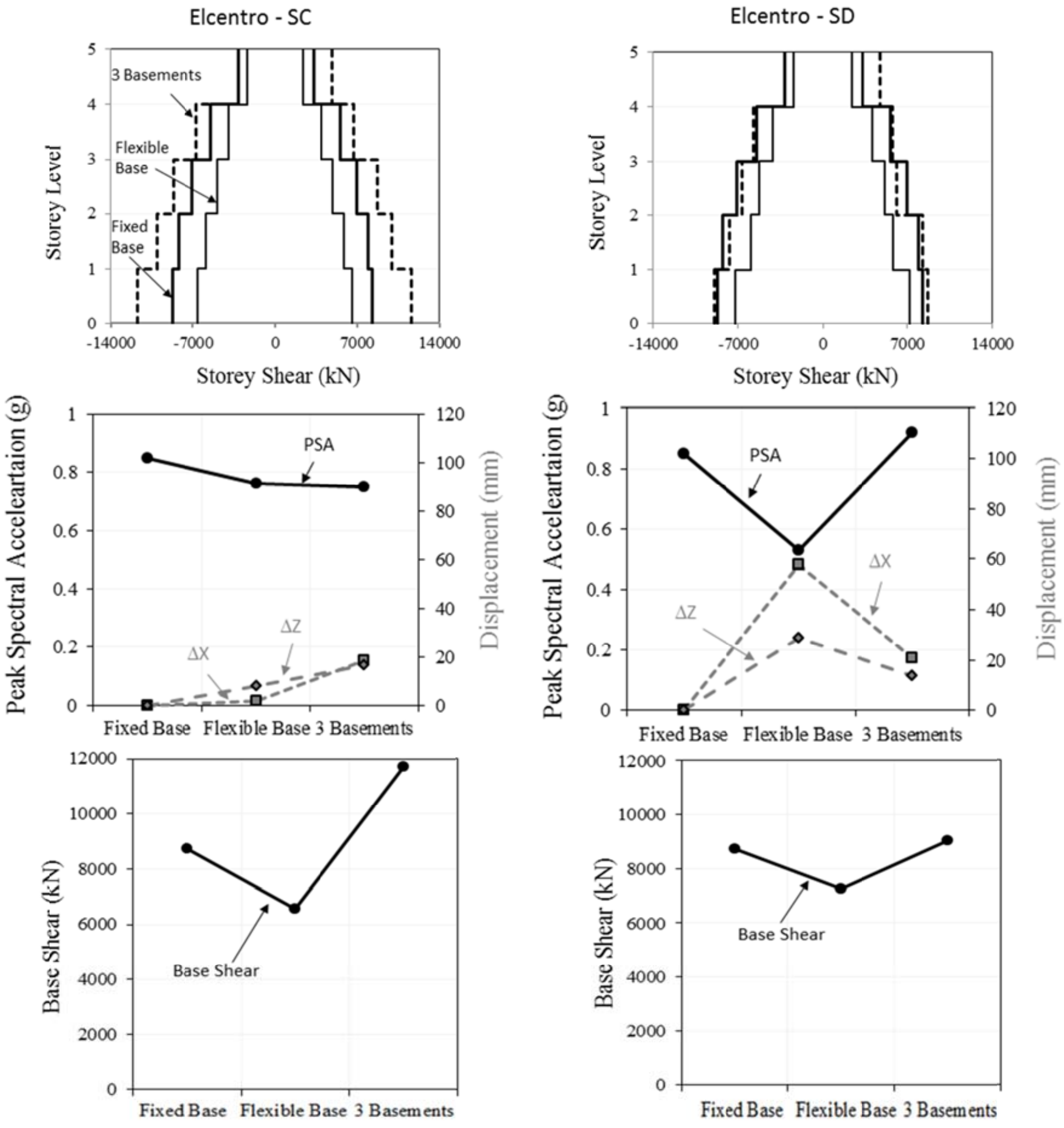
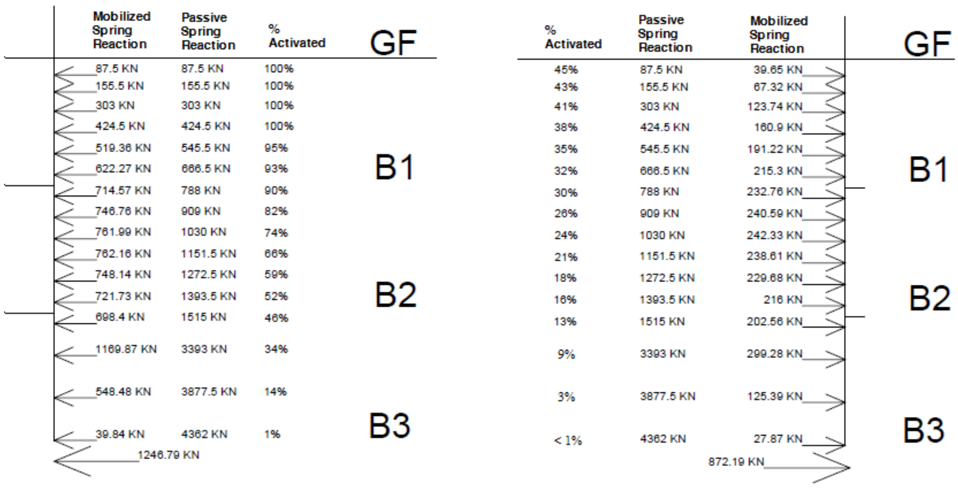


Fig. 8 Variation of envelope storey shear demands, attracted spectral acceleration, structure's vertical and horizontal displacement at ground level, and base shear at instant of peak load demand for 5-storey buildings subject to the El-Centro ground motion for (a) Soil Class C, and (b) Soil Class D



(a) El-Centro - Soil Class C (b) Pasadena - Soil Class C

Fig. 9 Mobilization of side soil resistance of 5-storey buildings with 3 basement floors

3.2 Effect of soil type on seismic response of 5-Storey buildings

Results from section 3.1 indicate that the two main factors that could affect the magnitude of the storey/base shear are the peak spectral accelerations that are attracted by the seismic shaking and the displacements that result from the shaking. Both of these factors are expected to be significantly affected by the type of soil. To illustrate the impact of the type of soil on the response of the structure, the analysis that was conducted in section 3.1 (5-storey building, El-Centro ground motion) was repeated for the case where the soil is classified as Class D (SD) and the results are presented in Fig. 8(b). As was the case for Soil Class C, results on Fig. 8(b) indicate that the storey shear decreased in the flexible base case (no basements) compared to the base case where the building was assumed to be fixed at the foundation level. The calculated reduction in the base shear was only 17% (compared to 25% for soil class C) although the reduction in the spectral acceleration was significantly larger for Soil Class D compared to Soil Class C. This could be explained by the large lateral displacements that were computed for the flexible 0 basement case in soil class D (around 60 mm) compared to the negligible displacement observed in the case of soil class C. The relatively large displacement in the case of soil Class D mobilizes a relatively large reaction from the lateral spring representing the foundation. This contributes to the total base shear for the flexible base case with no basements and limits the overall reduction in the base shear to a relatively small value (17%).

For the case with 3 basements and soil class D, the computed base shear was found to be almost the same as that of the fixed base case (slight increase of 3.3%). This slight increase in base shear in the flexible 3 basement case appears to be directly correlated with a slight increase in the peak spectral acceleration compared to the fixed base case. If this is the case, it could be inferred that the side soil reactions for the case of Soil Class D did not play a significant role in amplifying the total base shear as was the case in Soil Class C, despite the lateral wall displacement of 16 mm that was computed at ground level. This displacement of 16 mm could be considered to be relatively small (compared to 18 mm for soil class C) given that the lateral foundation spring stiffness for soil class D was found to be almost 4 times smaller than the stiffness for soil class C. This unexpected and relatively small displacement of 16 mm in soil class D could be attributed to the fact that some rigid body translation of the basement walls occurred in soil class D releasing part of the strain energy of the earthquake and resulting in smaller lateral displacements at the top of the wall at ground level. This rigid body translation is evidenced in the fact that the wall foundation for soil class D was found to move laterally by 4.5 mm (compared to 0.5 mm for soil class C). It should also be noted that soils in class D are softer and more compressible than soils in class C leading to lower spring stiffnesses for foundation and side soils and to lower ultimate passive lateral reactions for the side soil. As a result, for a given wall or foundation lateral displacement, it is expected that lateral reactions for soil class D will be smaller than the reactions for soil Class C, making the storey/base shears less sensitive to the movement of the foundation or the basement wall.

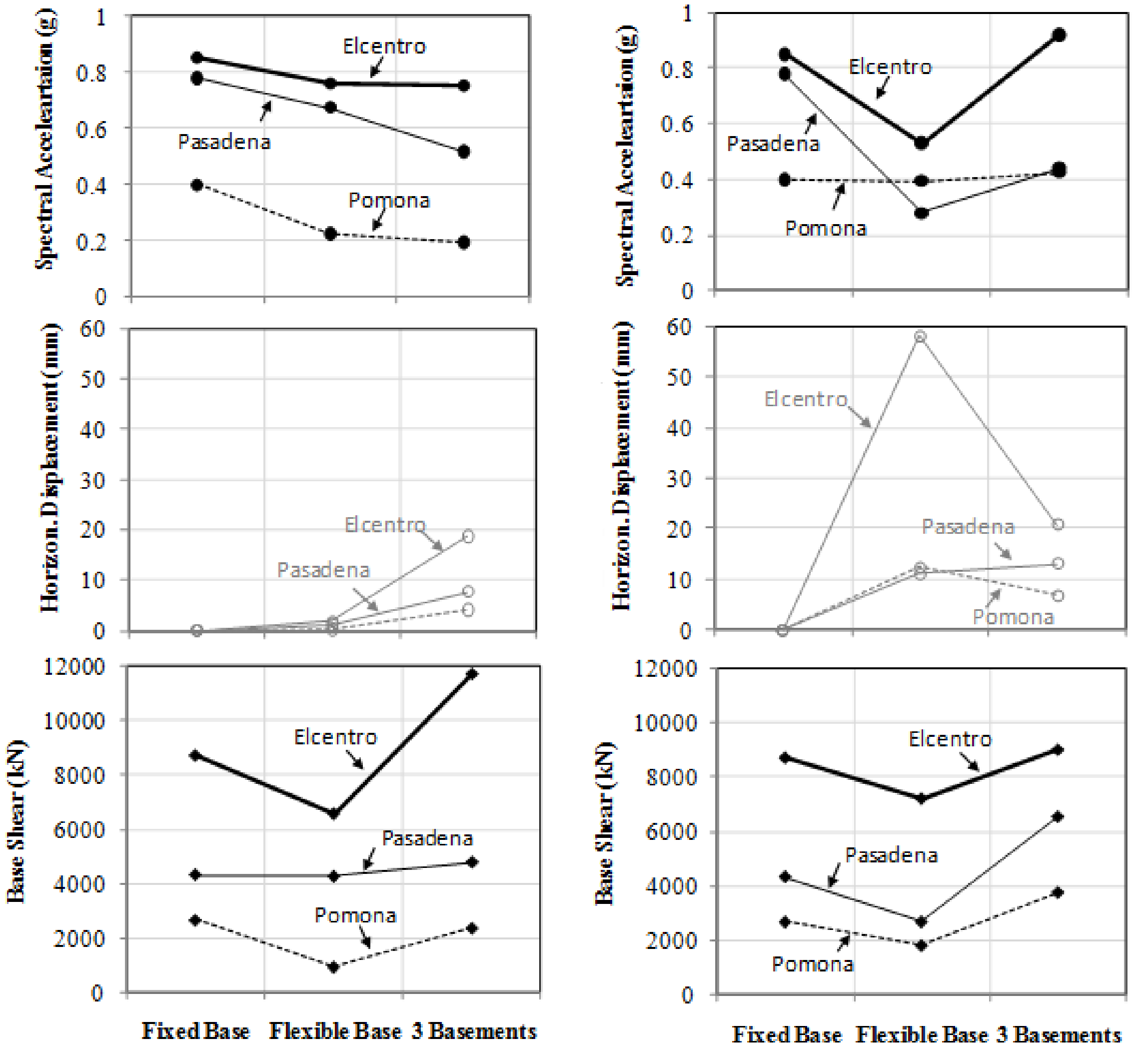
3.3 Effect of earthquake characteristics on seismic response of 5-storey buildings

Seismic excitations are generally characterized by their associated peak ground accelerations (PGA), frequency content, and duration of shaking. To investigate the effect of the input ground motion on the seismic response for the 5-storey buildings, the analysis that was conducted in sections 3.1 and 3.2 for the El-Centro ground motion on soil classes C and D was repeated for the Pasadena and Pomona earthquakes and the results are presented in Fig. 10.

For the case with Soil Class C, results on Fig. 10(a) indicate that (1) the computed base shear for the fixed base case was found to be the largest for the El-Centro ground motion and the smallest for the Pomona ground motion, as expected, (2) for all ground motions, the base shear was found to decrease in the flexible base case (no basements) compared to the fixed base case, with the percent decrease ranging from 1.3% (Pasadena) to about 64% (Pomona) and (3) when 3 basement floors with flexible foundations and side soil were included in the building models, the calculated base shear was found to be relatively close to the fixed base case with an increase of 10% for the Pasadena earthquake and a decrease of 10% for the Pomona earthquake. These percentages are smaller than the increase of 34% which was witnessed in the base shear of the 3-basement case for the more aggressive El-Centro shaking.

The above observations could be explained by analyzing the relationship between the changes in base shear and the corresponding variations in the computed peak spectral accelerations and the lateral displacements at ground floor. First, the consistent drop in the computed base shear of the fixed base case from the El-Centro ground motion (~8700 kN) to the Pomona ground motion (~2700 kN) is expected given the characteristics of the earthquakes (PGA, frequency, duration) and the spectral accelerations that were attracted by the building in each (Fig. 10(a)). Second, the drop in the peak spectral accelerations for the case of the flexible base with no basements is the reason for the observed reductions in the base shear compared to the fixed case. The reduction of 64% in the base shear of the Pomona earthquake could be directly correlated to a reduction of 45% in the peak spectral acceleration.

For the cases with 3 basement floors, the change in the computed base shear compared to the fixed base condition (+34% for El-Centro, +10% for Pasadena, and -10% for Pomona) did not correlate directly to the change in the spectral accelerations, which showed consistent drops for all earthquakes. However, a more consistent and relevant correlation seems to exist between the changes in base shear and the lateral displacements at the ground floor level whereby these displacements were the largest for the El-Centro earthquake (~18 mm) and the least for the



(a) Soil Class C (b) Soil Class D

Fig. 10 Variation of envelope storey shear demands, attracted spectral acceleration, structure's horizontal displacement at ground level, and base shear at instant of peak load demand for 5-storey buildings subject to the El-Centro, Pasadena, and Pomona ground motions for (a) Soil Class C, and (b) Soil Class D

Pomona earthquake (~3.5 mm). It is anticipated that the relatively small lateral displacements in the Pasadena and Pomona earthquakes resulted in the mobilization of relatively low lateral pressures in the soil springs acting on the basement walls, resulting in a more-or-less minor effect on the computed base shears. This is clearly illustrated in Fig. 9(b) which shows that the activation of lateral forces in the springs in the Pasadena earthquake was relatively low in comparison to the maximum passive forces that could be mobilized in soil Class C. For example, the maximum activation in the springs of the 1st basement was in the order 30 to 40% in the Pasadena earthquake compared to 90% to 100% in the El-Centro earthquake. This is illustrated graphically in Fig. 11

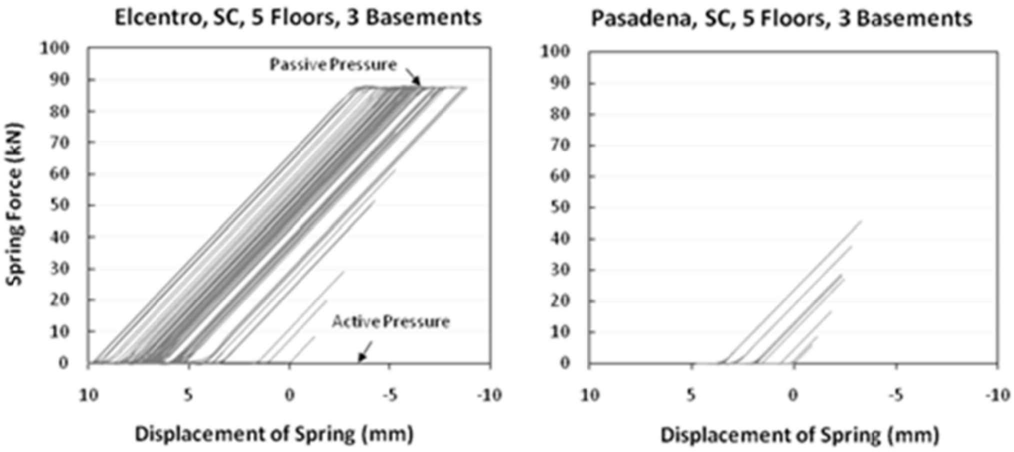


Fig. 11 Relationship between spring displacement and spring forces at the ground level for (a) El-Centro ground motion and (b) Pasadena ground motion for 5 storey buildings with 3 basements on Soil Class C

which shows how the spring forces are mobilized at ground level during the El-Centro and Pasadena earthquakes. The effects of the magnitude of the earthquakes and the number of cycles on the maximum spring force mobilized and the accumulation of displacements are clearly indicated in Fig. 11.

The above analysis pertains to soil class C. For soils in class D, results of the base shear for the flexible base case (0 basements) indicate consistent decreases (17% in El-Centro, 37% in Pasadena, and 31% in Pomona) compared to the fixed base case. For the case with 3 basements, and unlike the limited increase of 3.3% that was witnessed in the base shear for the El-Centro ground motion, the base shears in the Pasadena and Pomona earthquakes indicated increases of about 50% and 40%, respectively in soil class D compared to the fixed base case. These results indicate that the effects of incorporating SSI could be more significant in the cases of medium to low intensity ground excitations for five storey buildings on soil class D than similar structures on soil class C. This is associated with the greater structure displacements at ground level for buildings on medium dense soils. For high intensity earthquakes, both buildings on soil classes C and D undergo significant displacements at ground level to activate the passive soil resistance at top levels. For medium to low intensity earthquakes, the larger displacements in soil class D will activate lateral forces in excess of those activated in Class C leading to a higher relative increase in base shear. As an example, the percent activation of the passive lateral forces for the springs in the 1st basement ranges from 80% to 95% for the Pasadena (SD) compared to 30% to 40% for Pasadena (SC).

3.4 Effect of number of above ground stories on seismic response

The results of the analyses conducted in Sections 3.1 to 3.3 pertain to 5-storey buildings with relatively small fundamental periods of vibration (0.25 seconds for the fixed base case). At this relatively small fundamental period, the response spectra on Fig. 5 indicate that the fixed base case for the 5-storey building is located near the peak of the spectra for the three ground motions considered. This initial position on the response spectra makes the spectral acceleration sensitive to changes in the fundamental period of the building due to soil structure interaction. This sensitivity was clearly illustrated in the previous sections. For the 10 and 15 storey building scenarios, the fundamental periods for the fixed base are 0.75 seconds and 1.37 seconds, respectively. These relatively high values have two major impacts on the seismic response of the structures while incorporating SSI: (1) the relatively high periods lead to relatively small spectral accelerations compared to the 5-storey condition, and (2) the relatively high periods position the fixed base scenario on the descending less steep branch of the response spectra making the spectral accelerations less sensitive to increases in the period due to soil structure interaction in the flexible base and basement scenarios.

To quantify the effect of foundation flexibility and the presence of basements on the seismic response of relatively tall structures, the variation of the base shear for the ten and fifteen storey buildings on soil class C was calculated for the El-Centro, Pasadena, and Pomona earthquakes and plotted on Fig. 12 for the fixed case, flexible case, and the case involving basements (3 and 5 basements for the ten and fifteen storey buildings). For the flexible base scenario with 0 basements and as expected, the base shear was found to decrease compared to the fixed base case with computed reductions in the order of 8% to 33% for the 10 storey building, and 25% to 60% for the

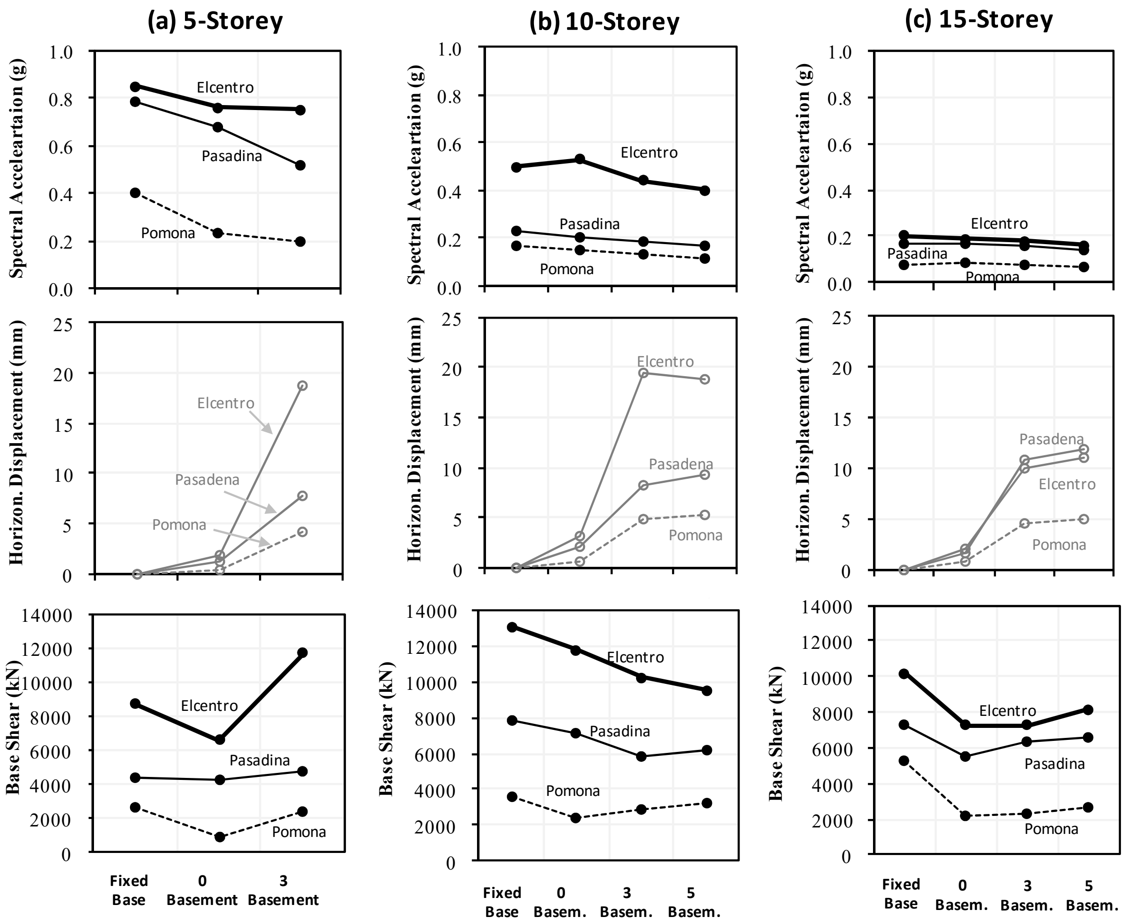


Fig. 12 Comparison between the seismic response of (a) five storey buildings, (b) 10 storey buildings, and (c) 15 storey buildings on Soil Class C and subject to the EL-Centro, Pasadena, and Pomona earthquakes

15 storey building for the different earthquake scenarios. The cases with 3 basements also indicated reduction in base shear compared to the fixed base case. The reductions were in the order of 18% to 26% for 10 storey buildings and 12% to 55% for 15 storey buildings with 3 basements. Very similar reductions of base shear were noted for the 5 basements option (10% to 27% for 10 storey buildings and 10% to 49% for the 15 storey building).

Results on Fig. 12 indicate that for tall structures (above 10 storeys), the assumption of a fixed base structure that is cropped at the ground floor level results in conservative estimates for the base shear. This is validated for all levels of shaking adopted in this study, irrespective of the number of basements. This does not apply for relatively short structures (5 storeys) since the case of 3 basements exhibited base shears that are in excess of those computed for the fixed base scenario.

4. Conclusions

The seismic performance of reinforced concrete shear wall buildings with underground stories is investigated in this study. Five, ten and fifteen storey buildings with underground stories ranging from zero to five basement floors are modeled. Two site conditions are considered: soil class C corresponding to “very dense soil or soft rock” and soil class D corresponding to “stiff soil”. The soil structure interactions effects are modeled using the multi-linear kinematic plastic link property of SAP2000. Three real event earthquake ground motions are simulated. The main objective is to check the effect of soil structure interaction both at foundation and basement wall levels. This is done via comparison to the standard practice case where the structure is cropped at ground floor level and designed irrelevant of the substructure parameters. The compared models included a flexible foundation spring model and another where the basement walls and side soil springs are included.

The foundation SSI effect is gauged by simulating the shallow footings interaction with the soil via equivalent linear springs representing the associated foundation stiffnesses in the different degrees of freedom. This added flexibility allows for energy dissipation which is translated as a decrease in the storey shear demands. This conclusion is accepted by most of the recent building codes, and it is encouraged to benefit from it.

However, for the models incorporating basements and the side soil springs, unexpected trends are discovered. It is shown that side soil structure interaction is not only significant for low rise stiff structures but even leads to governing design storey shear demands. Thus, the current standard practice of cropping the building at ground level, using full fixity conditions, and analyzing accordingly may lead to under-designed structures in the case of low rise stiff buildings. For taller and less rigid structures, the study shows that the fixed base analysis case provided conservative results as the envelope of the building storey shear demand decreases once SSI effects are incorporated. Therefore, while the current design practice will lead to a conservative design for high rise structures on soil classes C and D, significant design and cost optimizations could be achieved if the SSI effects are explicitly incorporated in the mathematical analysis model.

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