

EFFECT OF SOIL CONDITIONS ON THE RESPONSE OF REINFORCED CONCRETE TALL STRUCTURES TO NEAR-FAULT EARTHQUAKES

K. GALAL* AND M. NAIMI

Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, Quebec, Canada

SUMMARY

Characteristics of near-fault earthquakes (NFE) are particularly different from that of the far-field ones. Far-field ground motions are characterized by low peak ground acceleration (PGA) and high frequency, whereas near-fault ground motions have a high peak ground velocity (PGV) and long period pulse. Several recent earthquakes, e.g. 1992 Landers, 1994 Northridge, 1995 Kobe, and 1999 Chichi earthquake events, have caused substantial damage to near-fault flexible structures. The nonlinear dynamic behaviour of tall reinforced concrete (RC) frame structures subjected to NFE records is influenced by the characteristics of the foundation soil. The assumption of a fixed-base model for this type of structure might not adequately represent their seismic response. Therefore, the seismic performance evaluation analysis should take into account the soil–structure interaction (SSI). In this study, the seismic performance of a 20-storey and a 6-storey RC frame structures with fixed-base and flexible-base conditions is evaluated. The characteristics of the flexible-base models cover four types of soils, namely, soft soil, medium soil, stiff soil and a rock soil, as classified by the International Building Code. A set of 13 near-fault acceleration time histories recorded on the four types of soil from major earthquake events is selected for the analysis. Three criteria for scaling the records were considered, namely, same maximum spectral acceleration, same spectral acceleration at fundamental period of fixed-base model, and same spectral acceleration at fundamental period of flexible-base model. The analysis evaluates the effect of SSI on the dynamic behaviour by comparing the response of the flexible-base model to the fixed-base model of the structure when subjected to different earthquake records on a specific soil type. It is concluded that SSI effects could vary significantly according to the characteristics of the NFE record, the scaling criterion and the seismic performance indicator representing the SSI. This observation is valid for tall and low-rise structures. Copyright © 2007 John Wiley & Sons, Ltd.

1. INTRODUCTION

When foundation soil is subjected to seismic ground motion its response is dictated by the soil characteristic properties, the soil conditions and the characteristics of the earthquake. Wave propagation theory stipulates that soil layers modify the characteristics of the input seismic waves while they are travelling through it (Kobayashi *et al.*, 1986), therefore influencing the acceleration record. Several investigations have reported that near-fault earthquake (NFE) ground motions caused severe damage to structures (e.g., Singh, 1995; Attalla *et al.*, 1998). The severity in the structural response is related to the nature of the ground motion, which in turn is related to other parameters, among which are the geology and characteristics of foundation soil. Suzuki and Asano (2000) have studied the characteristics of the near-fault ground motions by using a stochastic approach and a synthesis simulation.

The near-fault ground motions are generally characterized by long-duration pulses that subject the structure to very high input energy at the early stage of the record (Makris and Roussos, 1998). Several

*Correspondence to: Khaled E. Galal, Department of Building, Civil and Environmental Engineering, Concordia University, 1455 Boul. de Maisonneuve West, EV 6.167, Montréal, Québec, H3G 1M8 Canada. E-mail: galal@bcee.concordia.ca

research studies (e.g., Krawinkler *et al.*, 2003; Alavi and Krawinkler, 2004) have reported that near-fault ground motions, characterized by equivalent pulses (Liao *et al.*, 2001), can cause high values of storey drift in structural members, forcing the structure to behave in the inelastic range (Iwan *et al.*, 2000) and leading to severe damage of the structure. Malhotra (1999) studied the effect of characteristics of pulse-like ground motions on the dynamic response of buildings, and found that near-fault ground motions with directivity effects tend to have a high PGV/PGA ratio, which influences their response characteristics.

For tall structures, the presence of a flexible foundation soil influences the nonlinear dynamic behaviour of the structure. Under strong shaking, the soil located in the vicinity of the structure will have a nonlinear behaviour with permanent deformations (Halabian and El-Naggar, 2002) causing changes in the natural period compared to the fixed-base condition (Avilés and Suárez, 2002; Dutta *et al.*, 2004). The soil–structure interaction (SSI) results in kinematic and inertial effects giving changes in the dynamic properties of the structure and the characteristics of the ground motion (Avilés *et al.*, 2002). If the foundation soil is stiff enough, the dynamic response of the structure is not influenced by the soil characteristics and the structure can be assumed as fixed at its base. If the structure is resting on a flexible medium, the dynamic response of the structure will be different from the case of a fixed base condition due to the interaction between the soil and the structure. Therefore a complete dynamic analysis to evaluate the performance level of a structure should consider the effect of SSI in the model. When SSI is considered, the ground motions imposed at the foundation of the structure are a function of the soil parameters, the travel path, the local site effects and the geometry of the surrounding soil. This is defined as the free field ground motion (Wolf and Deeks, 2004). A simplified procedure based on ductility demands is proposed by Avilés and Pérez-Rocha (2005) for design of yielding structures including SSI effects.

Performance-based seismic engineering is the modern approach to earthquake-resistant design. Seismic performance (*performance level*) is described by designating the maximum allowable damage state (*damage parameter*) for an identified seismic hazard (*hazard level*). Performance levels describe the state of a structure after being subjected to a certain hazard level as: fully operational, operational, life safe, near collapse, or collapse (Vision 2000, 1995; FEMA 273/274, 1997). Overall lateral deflection, ductility demand, and inter-storey drift are the most commonly used damage parameters. The five qualitative performance levels are related to corresponding five quantitative maximum inter-storey drifts (as a damage parameter) to be: <0.2%, <0.5%, <1.5%, <2.5%, and >2.5%, respectively.

In this study, the seismic performance represented by the inter-storey drift of a 20-storey RC frame structure with fixed-base and flexible-base conditions is evaluated. The characteristics of the flexible-base models covers four types of soils, namely, soft soil, medium soil, stiff soil and a rock soil as classified by the International Building Code (IBC, 2000). To simulate the impact of SSI on the foundation flexibility, the procedures for kinematics effects of ATC-40 recommendations (1996) are used. A set of 13 near-fault acceleration time histories recorded on the four types of soil from major earthquake events is selected for the analysis. Three criteria for scaling the records were considered, namely, same maximum spectral acceleration, same spectral acceleration at fundamental period of fixed-base model, and same spectral acceleration at fundamental period of flexible-base model. The analysis evaluates the effect of SSI on the dynamic behaviour by comparing the response of the flexible-base model to the fixed-base model of the structure when subjected to different earthquakes records on a specific soil type.

2. DESIGN CONSIDERATION OF THE REINFORCED CONCRETE BUILDINGS

Two RC moment-resisting frame (MRF) buildings are used in this study; a 20-storey building to represent tall structures and a 6-storey building to represent low-rise structures. The structures were designed according to the National Building Code of Canada (NBCC, 1995) and CSA (1994) using

seismicity characteristics of Victoria city in British Colombia, Canada, as a highly seismic zone in comparison to the rest of the country (El-Sheikh, 2002). The buildings have a three-bay by three-bay square plan. The floor-to-floor height of the storeys is 3.6 m. The total heights for the 20-storey building and the 6-storey building are 72 m and 21.6 m, respectively. Figure 1 shows the elevation of the two MRF and the cross-section properties of the columns and beams. The slab thickness is taken 180 mm to meet the minimum thickness requirement according to CSA (1994). In the original design, the structure models were assumed to be fixed at the base with no consideration of soil flexibility.

3. MODELLING SOIL–STRUCTURE SYSTEM

3.1 Superstructure parameters and model

Frames were designed according to the strong column–weak beam approach to achieve the desired beam hinging mechanism. This design approach requires that flexural resistances of the columns and the beams satisfy the following criteria:

$$\sum M_{rc} \geq 1.1 \sum M_{nb} \quad (1)$$

where $\sum M_{rc}$ is the sum of moments at the centre of the joint, corresponding to the factored resistance of the columns framing into the joint, and $\sum M_{nb}$ is the sum of moments at the centre of the joint corresponding to the nominal resistance of the beams and girders framing into that joint.

The weight of the structure is input as masses concentrated at the member joints. The total weight per floor per frame is 675 kN, which includes the weight of floor slab, exterior and interior partition walls and mechanical services. Thus, the weight at exterior nodes and interior nodes is taken as 112.5 kN and 225 kN, respectively. Beam–column joints are assumed to be rigid and are represented by rigid zones at the elements ends. The length of the rigid zone was assumed to be 0.4 of the depth of the columns and beams in each side of the joint.

3.1.1 Modelling of beams and columns

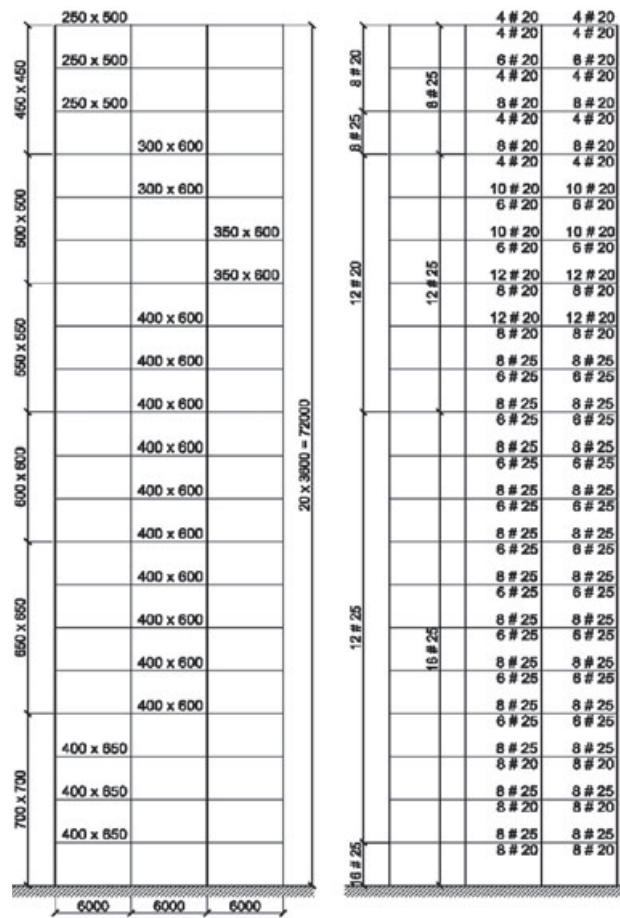
The buildings are idealized as two-dimensional frame models. The beams are modelled as linear elastic elements with two inelastic flexural springs (plastic hinges) for flexural deformation at each end, as shown in Figure 2(a). A trilinear pinching model (Figure 2b) is used to model the nonlinear rotation at each end. Shear deformation is assumed to be elastic.

The columns are modelled as an elastic element with two inelastic multi-spring elements at each end, as shown in Figure 3(a). The multi-spring elements are capable of simulating the axial and flexural nonlinear behaviours taking into account the coupling effect between flexure and axial force.

The trilinear/bilinear model SS3 shown in Figure 3(b) was used to represent the steel bars (Li, 1999). A bilinear skeleton curve with specified hysteretic parameters was used for the current analysis with the stress–strain properties provided for each steel bar. The parameters for the steel skeleton curve are listed in Table 1(a). The concrete behaviour was represented using the bilinear model CS2 (Li, 1999) shown in Figure 3(c). The model is simple with post-peak degrading envelope and tension-stiffening capabilities. The parameters for the concrete skeleton curve are listed in Table 1(b). Linear shear deformation was assumed.

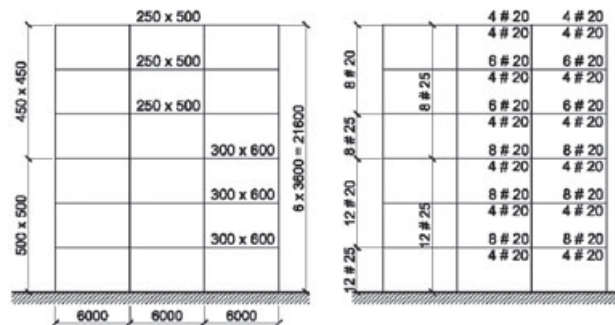
3.1.2 Material properties

The specified compressive strength of concrete was $f'_c = 25$ MPa, the specified yield strength of steel was $f_y = 400$ MPa and the concrete density was $\gamma_c = 24$ kN/m³. The modulus of elasticity of concrete was calculated according to clause 8.6.2.2 (CSA, 1994) as follows:



Cross-section dimensions Steel reinforcement details

(a)



Cross-section dimensions Steel reinforcement details

(b)

Figure 1. Elevation and cross-section properties of the studied MRFs (fixed-base model): (a) 20-storey building; (b) 6-storey building (El-Sheikh, 2002)

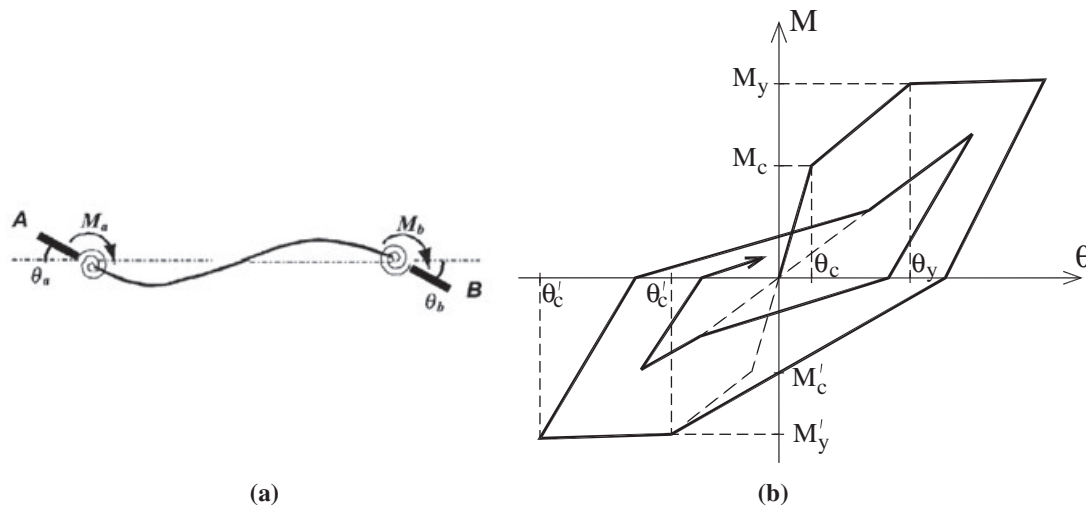


Figure 2. Modelling of beams: (a) uniaxial flexural spring model; (b) cross-peak trilinear pinching model at plastic hinges (Li, 1999)

Table 1(a). Parameters used for element SS3 (Li, 1999)

Parameter	Value
Skeleton curve parameters ν and κ	1.0
Post-yielding parameter β	0.01
Parameter ϕ to direct unloading	0
Unloading stiffness degrading parameter γ	0.2
Unloading control parameter θ	0.75

Table 1(b). Parameters used for element CS2 (Li, 1999)

Parameter	Value
Strain at maximum compressive strength	0.002
Compression post-peak residual/max. capacity ratio λ	0.2
Ultimate strain/strain at maximum compressive strength ratio μ	1.75
Post-peak unloading stiffness parameter γ	0.2

$$E_c = (3300\sqrt{f'_c} + 6900) \left(\frac{\gamma_c}{2300} \right)^{1.5} \quad (2)$$

3.2 SSI representation

The seismic response of a structure resting on soil flexible foundation requires good knowledge of soil parameters such as the soil impedance function, represented by the spring and damping coefficients. The effect of the supporting soil can be significant for the case of a tall flexible structure resting

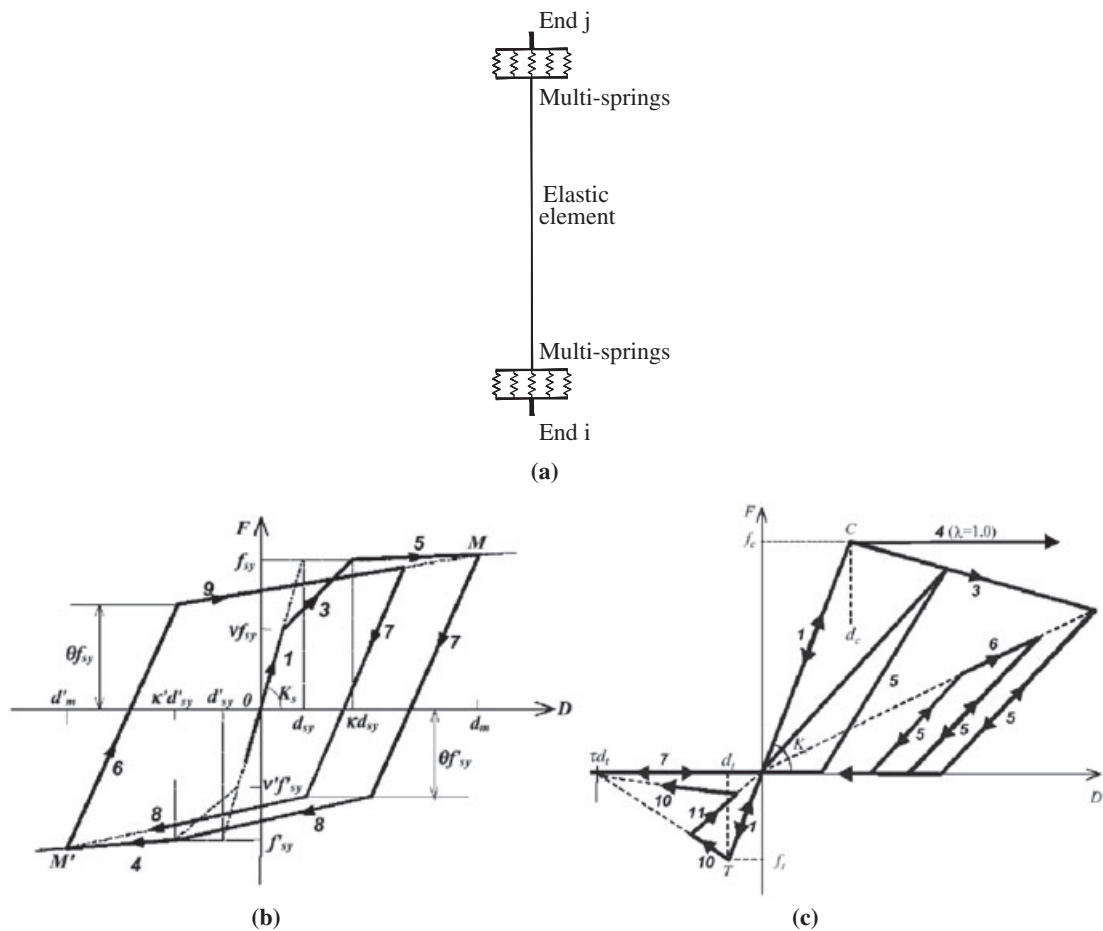


Figure 3. Modelling of columns: (a) multi-spring model; (b) hysteresis model for steel, SS3; (c) concrete material bilinear model, CS2. (Li, 1999)

on a deformable flexible soil leading to soil nonlinearity. At the vicinity of the structure foundation and under strong earthquake ground motion, the soil undergoes nonlinear deformations leading to an increase in the fundamental period. If the supporting soil is stiff enough, the response of the structure will not be influenced by the support condition; i.e., the structure can be assumed to be fixed at its base. If the foundation soil is less rigid it will have an influence on the overall structural response and SSI will take place. In this case, the flexibility of the supporting soil has to be accounted for.

The ground motion during an earthquake is dependent the soil parameters, the travel path, the local site effects and the geometry of the surrounding soil. This is defined as the free field ground motion. When soil is considered, the ground motions imposed at the foundation of the structure are influenced by its parameters. In order to account for the effect of SSI the suggested procedure of FEMA 440 (2005) is applied. The soil-structure models should be chosen cautiously in order to represent adequately the overall system. The soil physical characteristics are assumed as a lumped parameter system at the foundation level represented by the stiffness coefficients. Thus, a set of two translational and rotational springs is placed at the base of the ground floor columns to model the foundation soil. The

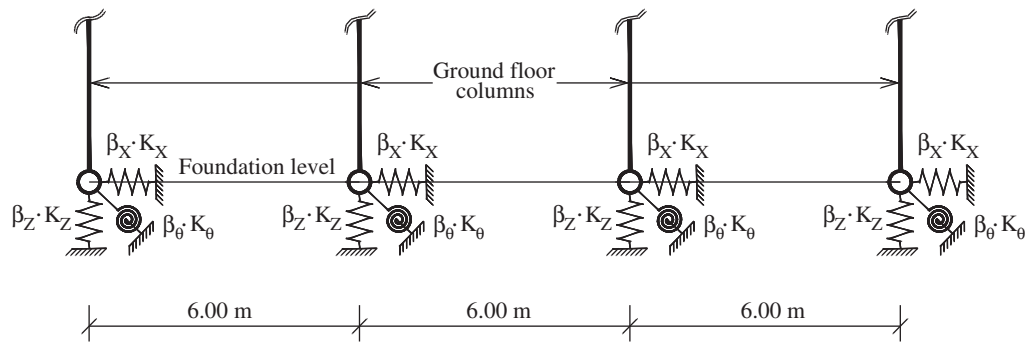


Figure 4. Modelling of the foundation supports

spring stiffness values are determined according to the shear wave velocity and Poisson's ratio of the geological medium that supports the structure.

Owing to the effect of SSI, the damping of the structure will be different from that of a fixed-base condition. The reason for this is that the presence of soil introduces additional damping in the soil–structure system in addition to the radiation damping. Thus, the energy of the vibrating system is dissipated into the soil medium through radiating waves (Mylonakis and Gazetas, 2000). The current study is mainly concerned with the investigation of the effect of soil conditions and the characteristics of the ground motions on the nonlinear dynamic performance of the structure through the estimation of the maximum inter-storey drift (ID_{max}). Thus, the effects of radiating damping on the response calculations are not included herein and it is assumed that only the material damping is present in the system. This is accounted for by using Rayleigh damping as a combination of the stiffness matrix and the mass matrix.

3.3 Soil parameters and model

The structures are assumed to be resting on a mat foundation of plan dimensions $21 \times 21 \text{ m}^2$ with thickness of 1 m and embedment of 2.5 m. The flexibility of the foundation soil is accounted for by using concentrated equivalent horizontal, vertical and rotational springs placed at the ground floor level of the structure as shown in Figure 4. This representation is found to be a satisfactory model for SSI (Wolf, 1985; FEMA 440, 2005). The procedure outlined in FEMA 440 (2005), developed by Gazetas (1991), is applied for the estimation of springs' stiffnesses. The stiffness expressions of the horizontal, vertical and rotational springs representing the soil medium are defined by K_X , K_Z and K_θ , respectively. To account for the embedment of the foundation, the spring stiffnesses are multiplied by the embedment factors β_X , β_Z and β_θ , respectively. Table 2 gives the expressions of the spring stiffnesses and their corresponding embedment factors.

The values of the stiffnesses of the springs are dependent on the mechanical characteristics of the soil material, the dimensions of the foundation, and its embedment depth. The mechanical characteristics of the foundation soil medium are represented by the effective shear modulus G , the mass density ρ_0 , and Poisson's ratio ν . At low strain, the maximum shear modulus G_0 is related to the shear wave velocity C_s according to the following expressions (ATC-40, 1996):

$$C_s = \sqrt{\frac{G_0}{\rho_0}} \quad (3a)$$

Table 2. Expressions for spring stiffnesses and their corresponding embedment factors (FEMA 440, 2005)

Mode	Stiffness coefficient	Embedment factor
Horizontal	$K_x = \frac{GL}{1-\nu} \left[0.73 + 1.54 \left(\frac{B}{L} \right)^{0.75} \right]$	$\beta_x = \left[1 + 0.095 \frac{D}{B} \left(1 + 1.3 \frac{B}{L} \right) \right] \left\{ 1 + 0.2 \left[\frac{(2L+2B)}{LB} d \right]^{0.67} \right\}$
Vertical	$K_z = \frac{GL}{2-\nu} \left[2 + 2.5 \left(\frac{B}{L} \right)^{0.85} \right]$	$\beta_z = \left[1 + 0.15 \left(\frac{2D}{B} \right)^{0.5} \right] \left\{ 1 + 0.52 \left[16 \frac{(D-d/2)(L+B)d}{BL^2} \right]^{0.4} \right\}$
Rotational	$K_\theta = \frac{G}{1-\nu} I_x^{0.75} \left(\frac{L}{B} \right)^{0.25} \left(2.4 + 0.5 \frac{B}{L} \right)$	$\beta_\theta = 1 + 2.52 \frac{d}{B} \left[1 + \frac{2d}{B} \left(\frac{d}{D} \right)^{-0.20} \left(\frac{B}{L} \right)^{0.50} \right]$

Note: G is the effective shear modulus (kN/m²); ν is Poisson's ratio; L is the length of the foundation (m); B is the width of the foundation (m); d is the thickness of the foundation (m); and D is the embedment depth of the foundation (m).

Table 3. Properties of the four considered soil types

Soil type (IBC)	Soil profile	Range of shear wave velocity, C_s	Average shear wave velocity (m/s)	Poisson's ratio, ν	Initial shear modulus, G_0 (kN/m ²) $\times 10^3$	Effective shear modulus $G = 0.42G_0$ (kN/m ²) $\times 10^3$
E	Soft soil	$C_s < 180$	160	0.25	4.4	1.84
D	Stiff soil	$180 < C_s < 360$	270	0.25	13.7	5.75
C	Very dense soil and soft rock	$360 < C_s < 750$	564	0.25	67.5	28.40
B	Rock	$750 < C_s < 1500$	1125	0.25	335.0	141.00

or

$$G_0 = \frac{\gamma}{g} C_s^2 \quad (3b)$$

where γ is the unit weight of the soil and g is the gravity acceleration.

This study covers four types of soils as classified by IBC (2000), namely, soil types E, D, C and B. Most of the design codes classify the soil types according to the ranges of the shear wave velocity C_s . Table 3 shows the properties of the four types of soil considered in this study as given by the IBC (2000). In representing the effect of SSI, the spring models used for the foundation soil are assumed to have compression resistance with no tension. The nonlinear behaviour of soil under dynamic cyclic strain can be represented by complex hysteretic models that have several control parameters, which are difficult to evaluate. In this study, the nonlinear behaviour of soil is simplified by assuming a degrading bilinear model with no tension for the vertical springs, as shown in Figure 5(a). The yielding force of the springs, F_y , is defined by the ultimate soil bearing capacity, q_{ult} , multiplied by the effective foundation area. The following formula given by ATC-40 (1996) is used for evaluating q_{ult} :

$$q_{ult} = c \cdot N_c \cdot q_c + \gamma \cdot D \cdot N_q \cdot q_q + \frac{1}{2} \gamma \cdot B \cdot N_\gamma \cdot q_\gamma \quad (4)$$

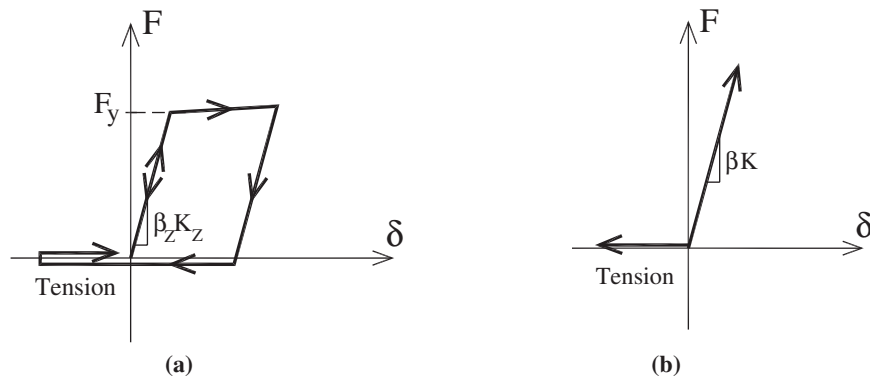


Figure 5. Modelling of the: (a) vertical; and (b) horizontal and rotational springs (Li, 1999)

Table 4. Properties of the vertical springs of the exterior and interior supports

Soil type (IBC)	Soil total unit weight, γ (kN/m ³)	Ultimate bearing capacity, q_{ult} (kN/m ²)	F_y for exterior supports (kN) $\times 10^3$	F_y for interior supports (kN) $\times 10^3$
E	16.81	135.72	3.66	4.89
D	18.41	598.75	16.20	21.60
C	20.83	958.00	25.90	34.50
B	26.00	1317.25	35.60	47.40

where c is the cohesion of soil; N_c , N_q , and N_γ are the bearing capacity factors functions of soil friction angle ϕ ; q_c , q_q , and q_γ are shape factors; D is the embedment depth of the foundation; and B is the width of the foundation.

The values of the ultimate bearing capacity are taken from Table 10-7 of ATC-40 (1996). The estimated values of q_{ult} are average values. The post-yield stiffness is assumed as 0.1 of the initial stiffness. Table 4 shows the properties of the vertical springs of the exterior and interior supports. The horizontal and rotational springs are composed of two counteracting springs that are assumed to have a linear elastic behaviour with no tension (Figure 5b).

4. INPUT GROUND MOTION

Most of the NFE events have provided strong evidence that the level of destructive damage is directly related to the nature of the fault rupture and its orientation with respect to the damaged site. The near-fault site is typically assumed to be at distance less than 50 km from the fault. At the vicinity of the fault rupture, the ground motions have different characteristics from those located at a far distance and they are influenced by the rupture mechanism that is called directivity. When there is a fault rupture, the generated waves propagate in a certain direction at a certain speed close to the shear wave velocity, causing most of the energy to arrive at a site within a very short period of time. Therefore, the direction of the fault rupture affects the site located in the vicinity. For the near-fault ground motions with forward directivity, the wave front arrives at a site as pulse like motion characterized by large amplitude. The investigation should include the effects of the characteristics of the near-fault ground motion for a site having particular soil conditions. For this purpose, a set of 13 NFE

Table 5. NFE ground motions recorded on soil types E, D, C and B

Soil type	Earthquake	Date	Station and component	M	R (km)	PGA (g)	PGV (cm/s)
E	Imperial Valley	1979/10/15	Impall/H-E03140	6.5	9.3	0.266	46.8
	Kobe	1995/01/16	Kobe/TAK000	6.9	0.3	0.611	127.1
	Westmorland	1981/04/26	Westmorl/WLF225	5.8	0–15	0.199	16.4
D	Cape Mendocino	1992/04/25	Capemend/PET090	7.1	9.5	0.662	89.7
	Central California	1960/01/20	Ctrcalif/B-HCH271	—	14.9	0.063	3.6
	Chichi, Taiwan	1999/09/20	Chichi/CHY101-N	7.6	11.14	0.44	115.0
C	Cape Mendocino	1992/04/25	Capemend/RIO270	7.1	18.5	0.385	43.9
	Coyote Lake	1979/08/06	Coyotelk/G06230	5.7	3.1	0.434	49.2
	Duzce, Turkey	1999/11/12	Duzce/1058-E	7.1	0.9	0.111	14.2
B	Anza (Horse Cany)	1980/02/25	Anza/PFT135	4.9	13.0	0.131	5.1
	Cape Mendocino	1992/04/25	Capemend/CPM000	7.1	8.5	1.497	127.4
	Northridge	1994/01/17	Northr/PUL194	6.7	8.0	1.285	103.9
	N. Palm Springs	1986/07/08	Palmspr/WWT180	6.0	7.3	0.492	34.7

acceleration time histories recorded on different soil types is used to assess the nonlinear dynamic response of the 20-storey and 6-storey frame structures including SSI. The records were retrieved from the PEER web site (PEER, 2005). Table 5 shows the properties of the NFE records.

The records were divided into four groups with respect to the type of soil. The four soil types—E, D, C and B—are specified with respect to a given range of the shear wave velocity, C_s , shown in Table 3. In order to have a unified characteristic of the input motions, three scaling criteria are used for the records. The first set corresponds to scaling the earthquake records on the basis of maximum pseudo-spectral acceleration ($S_{a,max}$), as given by the International Building Code (IBC, 2000). The $S_{a,max}$ of the records are scaled to match those of the IBC code for the corresponding soil profile, as shown in Figure 6. The second set is obtained by scaling the records such that the spectral acceleration of the record matches that of the IBC at the computed period of the fixed-base (i.e., with no springs for SSI representation) structure model, $S_a(T_{fixed})$, as shown in Figure 7. The third set is obtained by scaling the records such that the spectral acceleration of the record matches that of the IBC at the computed period of the structure model with flexible-base condition (i.e., with springs that represent SSI), $S_a(T_{flexible})$, as shown in Figure 8.

5. FORMULATION

The soil–structure system is subjected to NFE ground motion applied at the base level in the x -direction as an acceleration time history. The nonlinear time history dynamic analysis is performed with the CANNY99 (Li, 1999) program using the implicit step-by-step Newmark β method, with $\beta = 0.25$ and $\gamma = 0.5$. The equilibrium equations of the interaction system in the time domain at time $t + \Delta t$ can be written in matrix form as

$$[M]\{\Delta \ddot{x}\}_{t+\Delta t} + [C]\{\Delta \dot{x}\}_{t+\Delta t} + [K]\{\Delta x\}_{t+\Delta t} = -[M]\{\Delta a\}_{t+\Delta t} + \{\Delta f\}_{t+\Delta t} + \{F_t'\}_t \quad (5)$$

where $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices, respectively. The incremental quantities represented by $\{\Delta \ddot{x}\}_{t+\Delta t}$, $\{\Delta \dot{x}\}_{t+\Delta t}$, $\{\Delta x\}_{t+\Delta t}$ and $\{\Delta a\}_{t+\Delta t}$ are the vectors of acceleration, velocity, displacement and input acceleration respectively. The incremental external load vector is represented

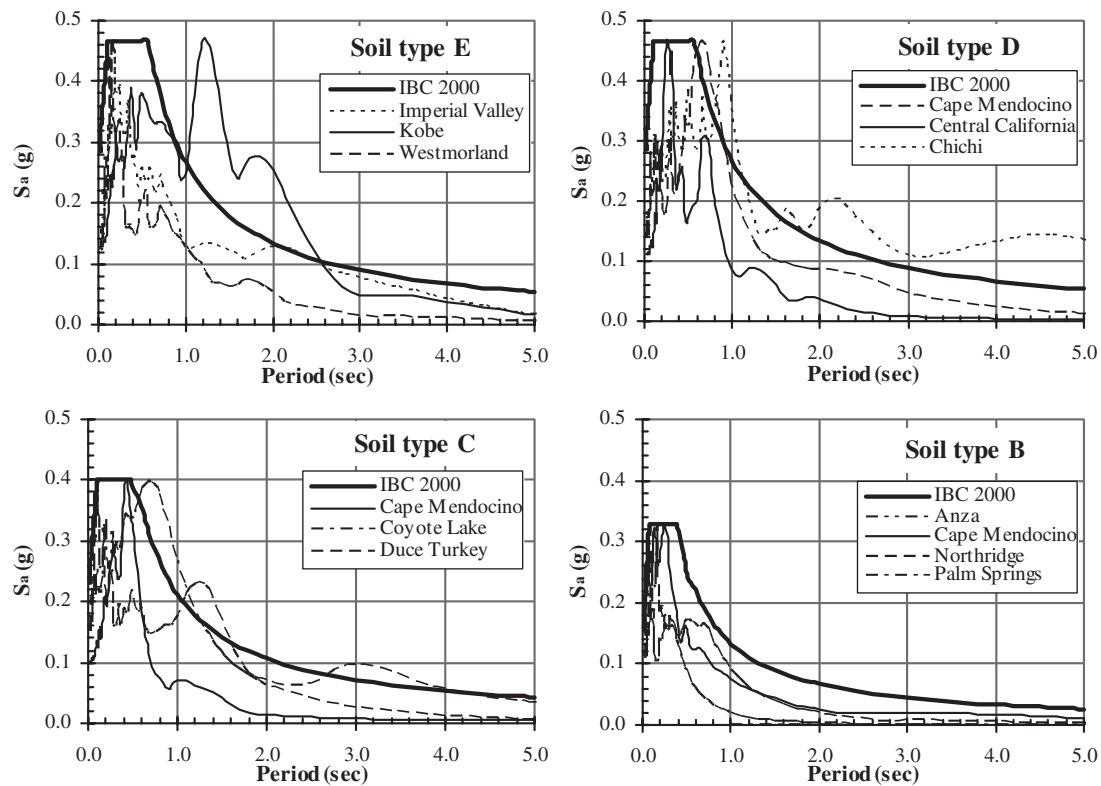


Figure 6. Spectral acceleration of the NFE records scaled to match $S_{a,max}$ of the IBC (2000) for soil types E, D, C and B

by $\{\Delta f\}_{t+\Delta t}$. At time t the unbalance force vector given by $\{F_t^J\}_t$ is expressed in terms of the inertia force vector, the external load vector, the total damping vector and the restoring force vector (Li, 1999).

6. RESULTS AND DISCUSSION

Results of the nonlinear time history analysis of the 20-storey and 6-storey structures with fixed- and flexible-base models in the form of the fundamental period, displacement time history, distribution of the shear force and the inter-storey drift were obtained. The following is a discussion of the results.

6.1 Free vibration analysis

The eigenvalue analysis is conducted to demonstrate the effect of including the soil parameters in predicting the fundamental period of the structure. The fundamental period of the fixed-base and flexible-base models were computed for both structures. Tables 6(a) and 6(b) show the elongation of the fundamental period of the flexible-base model for different types of foundation soil. The soft soil condition (soil type E) gives the highest fundamental period compared to the other cases, showing the importance of including soil parameters that lead to a lengthening of the period. The stiff soil profile (type D) and the very dense soil (type C) result in higher values of the fundamental period of the

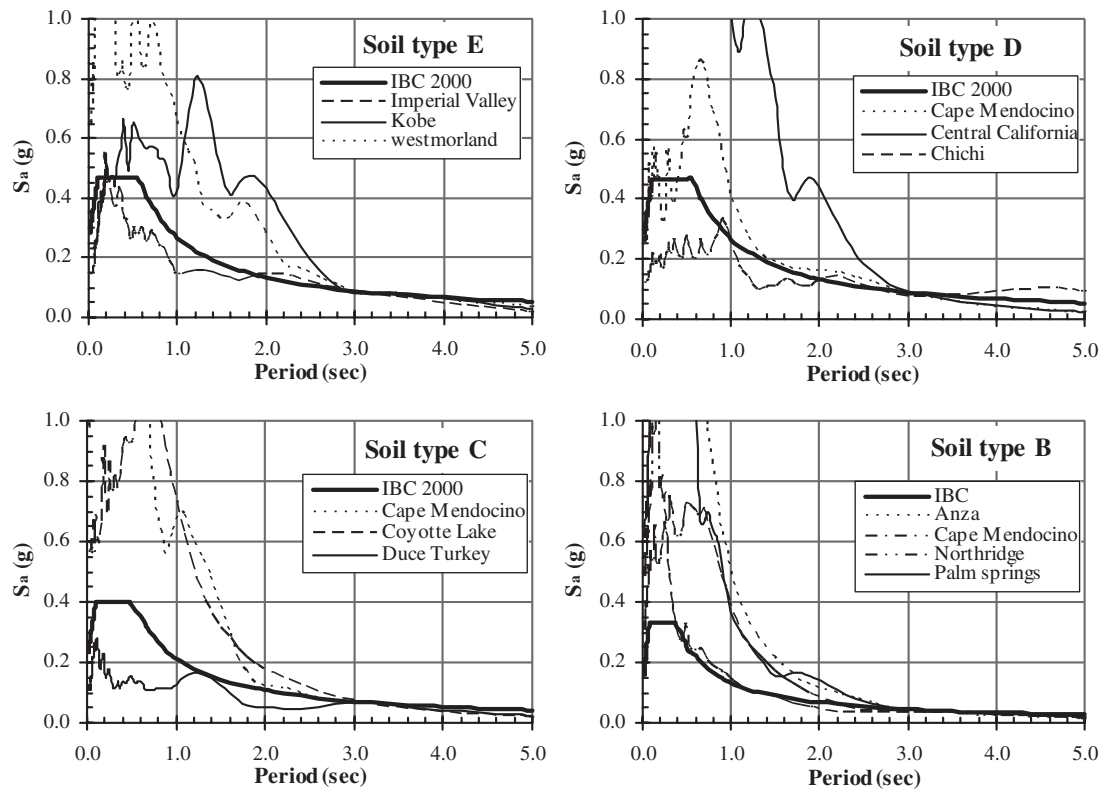


Figure 7. Spectral acceleration of the NFE records scaled to match $S(T_{\text{fixed}})$ of the IBC (2000) (case of 20-storey building) for soil types E, D, C and B

flexible-base model, whereas the case of rock material (type B) results in an almost identical value of the period compared to the fixed-base model, indicating that the foundation soil medium is rigid enough. This shows that for soil type B there is no SSI effect and the structure can be modelled using a fixed-base condition. From Tables 6(a) and 6(b), it can be seen that the effect of considering SSI on the fundamental period of the structure is higher for tall buildings than for low-rise buildings.

6.2 Analysis of 20-storey building (records scaled to $S_{a,\text{max}}$)

The response of the fixed-base and flexible-base models of the 20-storey building when subjected to a set of NFE records having the spectral accelerations shown in Figure 6 for soils types E, D, C and B, as classified by the IBC (2000) code, is evaluated. The results are shown in Figures 9–14 and Table 7. The following subsections discuss these results.

6.2.1 The influence of SSI on roof displacement history

Figures 9–12 show the displacement time histories of the top floor for models with fixed-base and flexible-base conditions representing the four soil types E, D, C and B.

For soil type E (soft soil) profile, it can be seen that SSI affects the response of the overall structural system. Figure 9(a) shows that the flexible-base model exhibits permanent roof displacement for

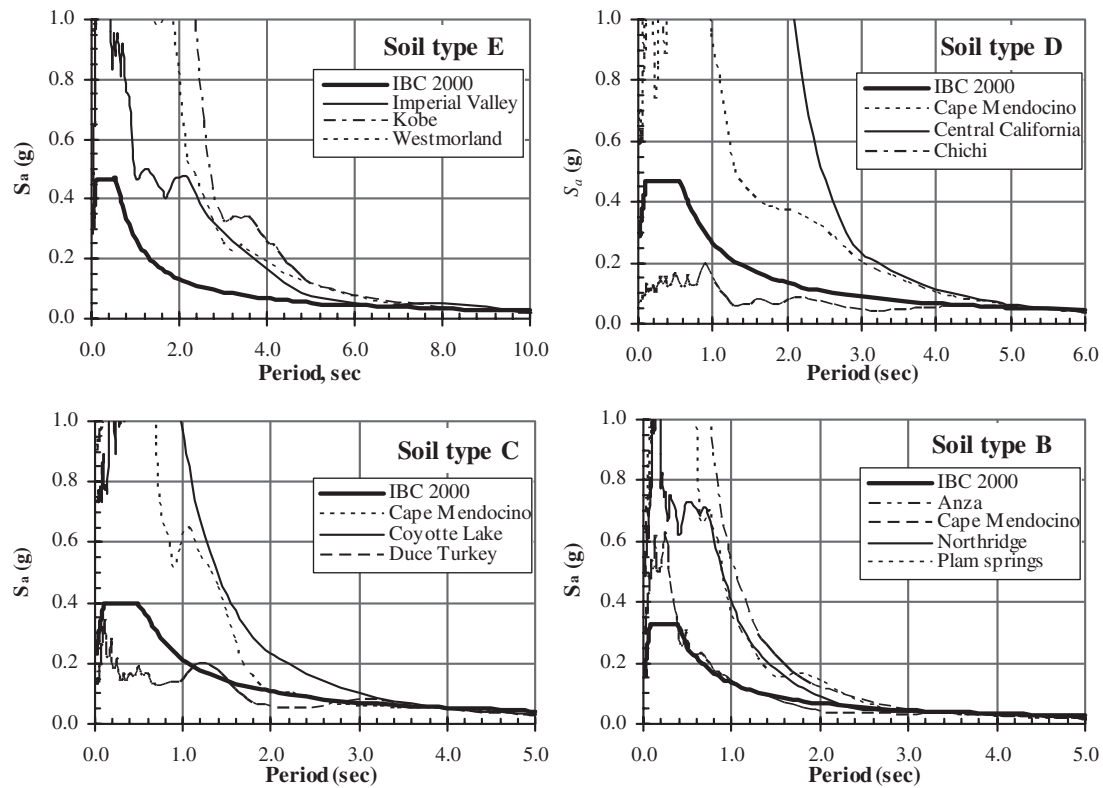


Figure 8. Spectral acceleration of the NFE records scaled to match $S(T_{\text{flexible}})$ of the IBC (2000) (case of 20-storey building) for soil types E, D, C and B

Table 6(a). Fundamental period, T , of the 20-storey building

Soil type	Fixed-base model (s)	Flexible-base model (s)	Ratio of periods ($T_{\text{fixed}}/T_{\text{flexible}}$)
E	3.28	9.74	0.34
D	3.28	5.70	0.57
C	3.28	3.93	0.83
B	3.28	3.44	0.95

Table 6(b). Fundamental period, T , of the 6-storey building

Soil type	Fixed-base model (s)	Flexible-base model (s)	Ratio of periods ($T_{\text{fixed}}/T_{\text{flexible}}$)
E	1.40	2.02	0.69
D	1.40	1.62	0.86
C	1.40	1.45	0.96
B	1.40	1.40	0.99

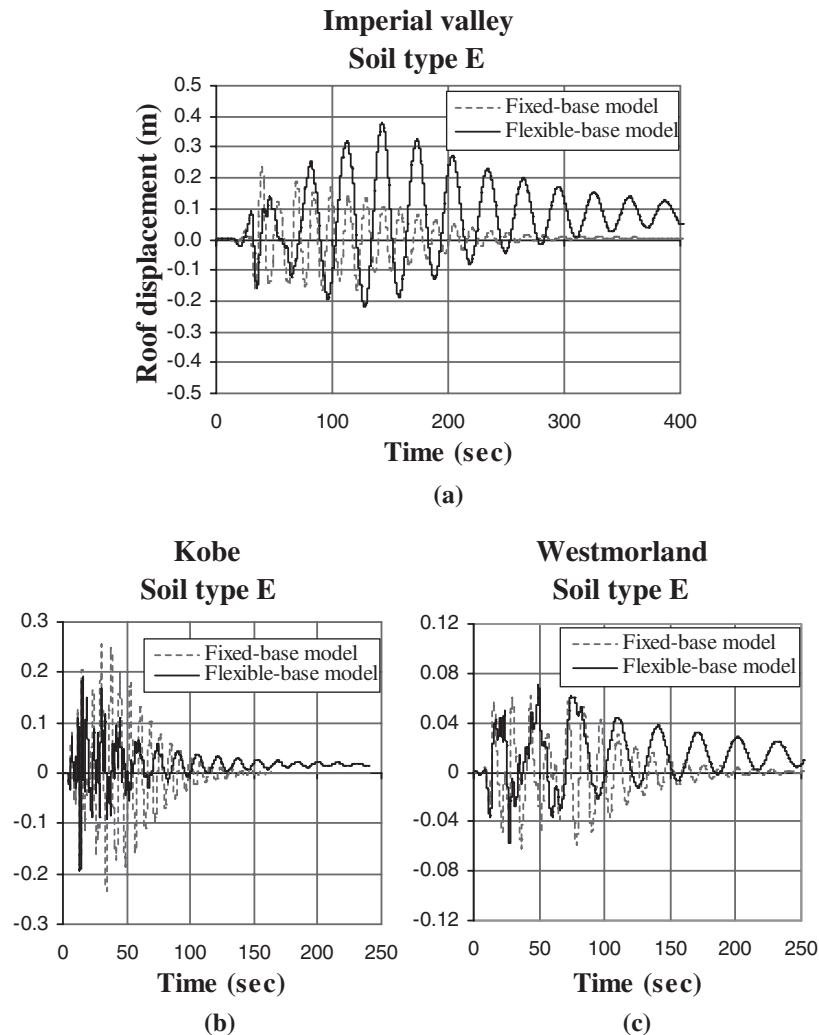


Figure 9. Displacement time-history response of the top floor of 20-storey building on soil type E subjected to earthquakes: (a) Imperial Valley; (b) Kobe; (c) Westmorland

the Imperial Valley ground motion. On the other hand, Figure 9(b) and (c) show negligible differences in the roof response of the fixed and flexible models for Kobe and Westmorland earthquakes.

For soil type D (stiff soil) profile, Figure 10(a)–(c) show that the SSI effects are significant through the presence of permanent roof deformations in the flexible-base model. Of particular interest is the case of Chichi earthquake (Figure 10(c)), which causes considerably larger roof displacement amplitude for the flexible-base model compared to the fixed-base model, indicating that SSI is imposing higher roof displacement demands. This can be attributed to the high spectral accelerations of the records at long period range compared to the Cape Mendocino and Central California records, even though the three records have the same $S_{a,max}$ (Figure 6).

On the other hand, the top-floor response of the 20-storey frame subjected to NFE records corresponding to soil types C (very dense soil profile) and B (rock), shown in Figures 11 and 12, indicate

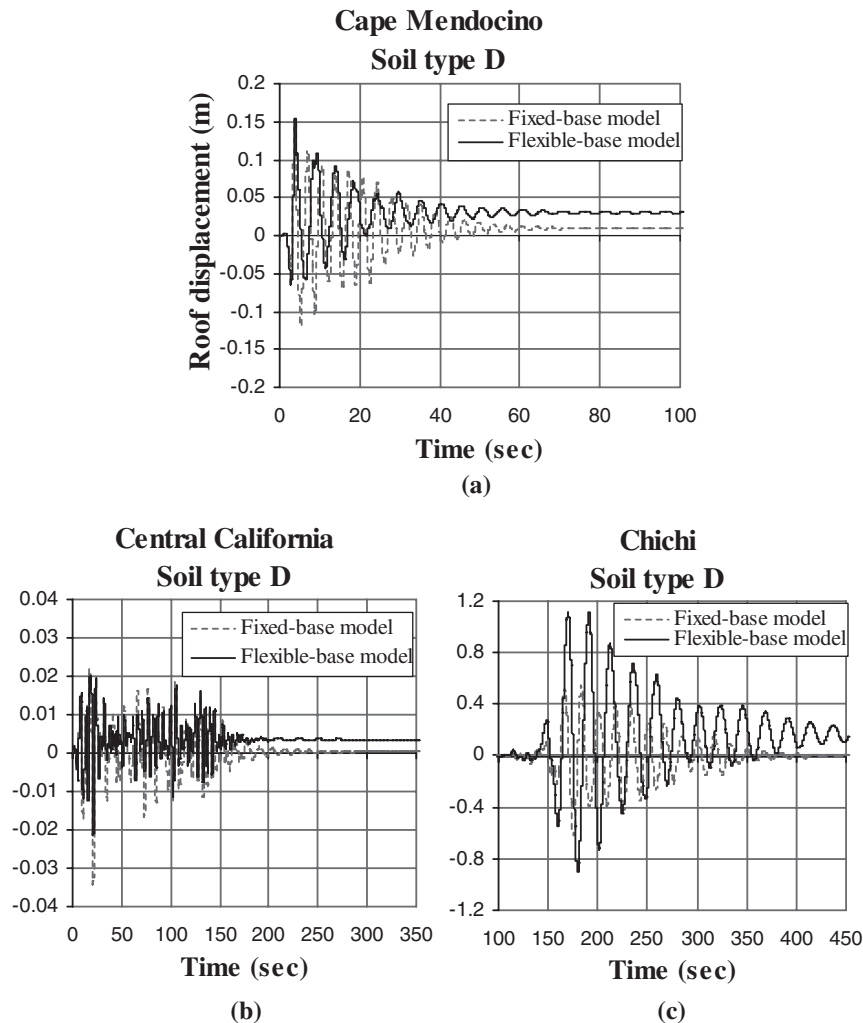


Figure 10. Displacement time-history response of the top floor of 20-storey building on soil type D subjected to earthquakes: (a) Cape Mendocino; (b) Central California; (c) Chichi

that the effects of SSI are not significant. In both cases, the flexible-base model shows small amounts of permanent top-floor deformations.

From the above, it can be concluded that the lateral displacement response of a flexible frame structure is highly sensitive to the soil conditions and the characteristics of the NFE record.

6.2.2 Influence of SSI on inter-storey drift and shear distribution

Figures 13 and 14 show the distribution of the inter-storey drifts and the shear force of the 20-storey building for fixed-base and flexible-base models when subjected to NFE records.

Figure 13 shows the distribution of the inter-storey drift ratio (ID) over the normalized height of the 20-storey frame for fixed-base and flexible-base models subjected to NFE records that are scaled to match the maximum spectral acceleration ($S_{a,max}$) for soil types E, D, C and B. From the figure it

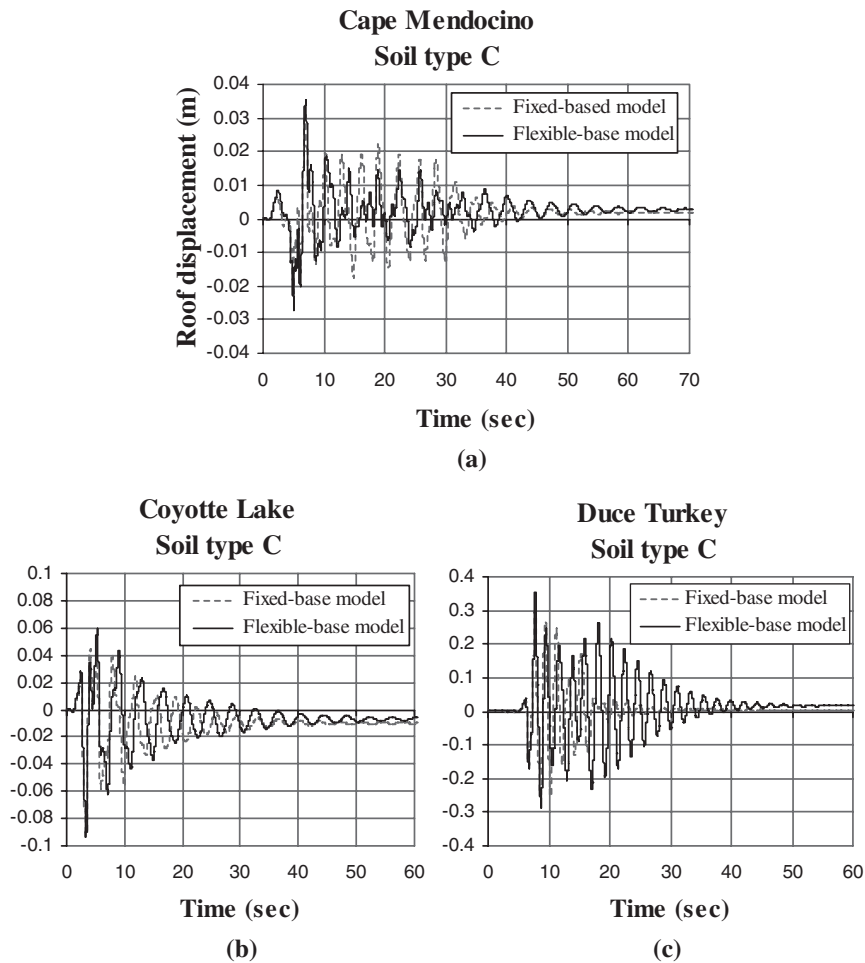


Figure 11. Displacement time-history response of the top floor of 20-storey building on soil type C subjected to earthquakes: (a) Cape Mendocino; (b) Coyotte Lake; (c) Duce Turkey

can be seen that the SSI, represented by the change in the inter-storey drift demand of the flexible-base model compared to the fixed-base model, is more pronounced in soil types E and D, and has less and negligible effects in soil types C and B, respectively. Although NFE records are scaled to the same $S_{a,max}$ that matches the $S_{a,max}$ of the IBC (2000) for the studied type of soil, it can be seen from the figure that the SSI is higher for particular NFE records (e.g., Imperial Valley record for soil type E and Chichi record for soil type D) and negligible for others. This implies that the effect of SSI on the inter-storey drift performance parameter of the flexible structures is highly dependent on the characteristics of the NFE record.

It is interesting to note that the increase in inter-storey drift due to SSI for both Imperial Valley and Chichi records is characterized by a more uniform ID along the height with higher ID values at the lower-level storeys when compared to the fixed-base model. This can be attributed to the lumped rotation at the lower floor of the building.

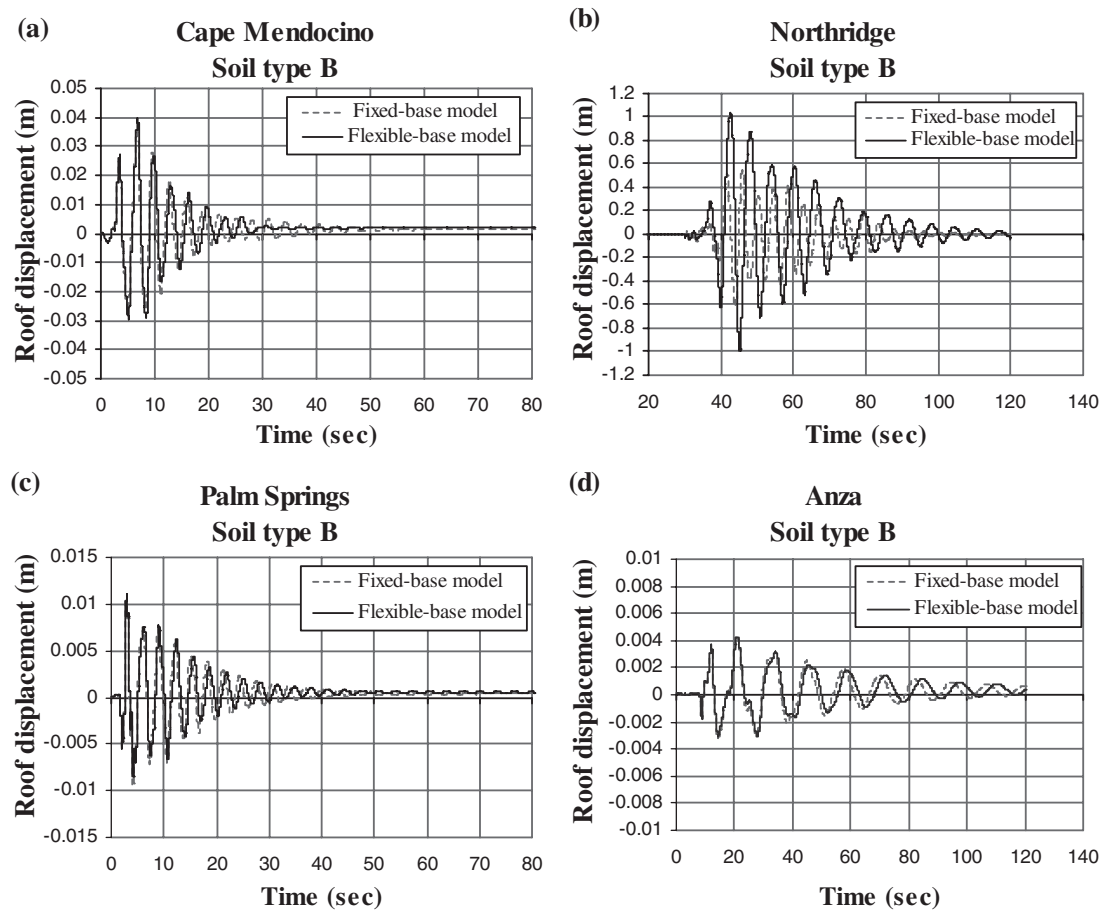


Figure 12. Displacement time-history response of the top floor of 20-storey building on soil type B subjected to earthquakes: (a) Cape Mendocino; (b) Northridge; (c) Palm Springs; (d) Anza

Figure 14 shows the ratio of the distribution of the maximum storey shear ($V_{i,max}$) to the maximum base shear ($V_{base,max}$) over the normalized height of the 20-storey frame for soil types E, D, C and B. From the figure, it can be seen that the SSI represented by the reduction in the maximum base shear of the flexible-base model compared to the fixed-base model reduces with increasing the soil stiffness (i.e., from soil type D to soil type B). For flexible soil types E and D, it can be seen that the SSI effects on the maximum base shear is dependent on the characteristics of the NFE record, even though the records are scaled to the same $S_{a,max}$.

6.3 Influence of scaling criteria on SSI effects

Table 7 shows the influence of scaling criteria of the NFE record on the SSI effects of the 20-storey frame for soil types E, D, C and B. The three scaling criteria described previously and shown in Figures 6–8 are examined. The SSI effects for each scaling criteria is evaluated using two indicators, namely, ID_{SSI} and V_{SSI} , which are defined as the ratios of the maximum inter-storey drift and base shear of the flexible- to fixed-base models, respectively.

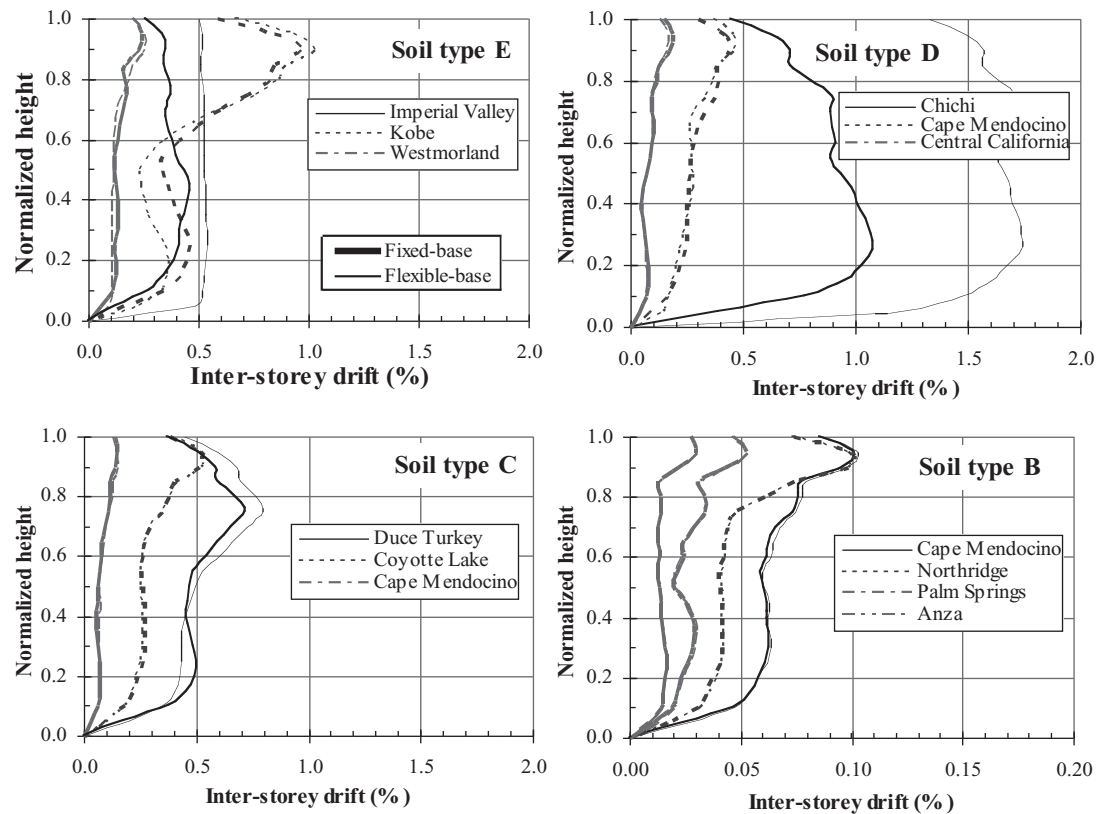


Figure 13. Distribution of inter-storey drift ratio over the normalized height of the 20-storey building for fixed-base and flexible-base models subjected to NFE records (scaled to match $S_{a,max}$ of IBC 2000) for soil types E, D, C, and B

From the table, it can be seen that SSI effects vary significantly according to the characteristics of the NFE record. This observation is more pronounced in less stiff soil, i.e., soil types E, D and C, and diminishes for soil type B. On the other hand, it can be seen from the table that the SSI is sensitive to both the scaling criterion and the indicator representing the SSI (i.e., ID_{SSI} or V_{SSI} ratios). For example, for soil type E, the Imperial Valley earthquake has the highest SSI effects on the building according to both ID_{SSI} and V_{SSI} ratios, if the $S_{a,max}$ scaling criterion is chosen. Yet, for the same soil type, Westmorland earthquake gives the highest SSI effect if ID_{SSI} ratio is considered and the second highest SSI effect if V_{SSI} ratio is considered. Moreover, it can be seen from the table that there are some cases where considering the SSI can result in a beneficial, i.e., unconservative, predicted performance in the analysis of a tall building subjected to a particular NFE record. This can be seen in the case of the Central California earthquake recorded on soil type D. A similar observation is reported by Mylonakis and Gazetas (2000).

Therefore, it can be concluded that in order to evaluate the seismic performance of flexible structures on flexible soil in the near-fault region, it is recommended to conduct a site study to determine a design ground motion record or spectrum for the specific site soil conditions. On the other hand, attention should be paid to whether considering SSI is beneficial or detrimental to the seismic performance of the structure when subjected to a particular ground motion. Consequently, in the former

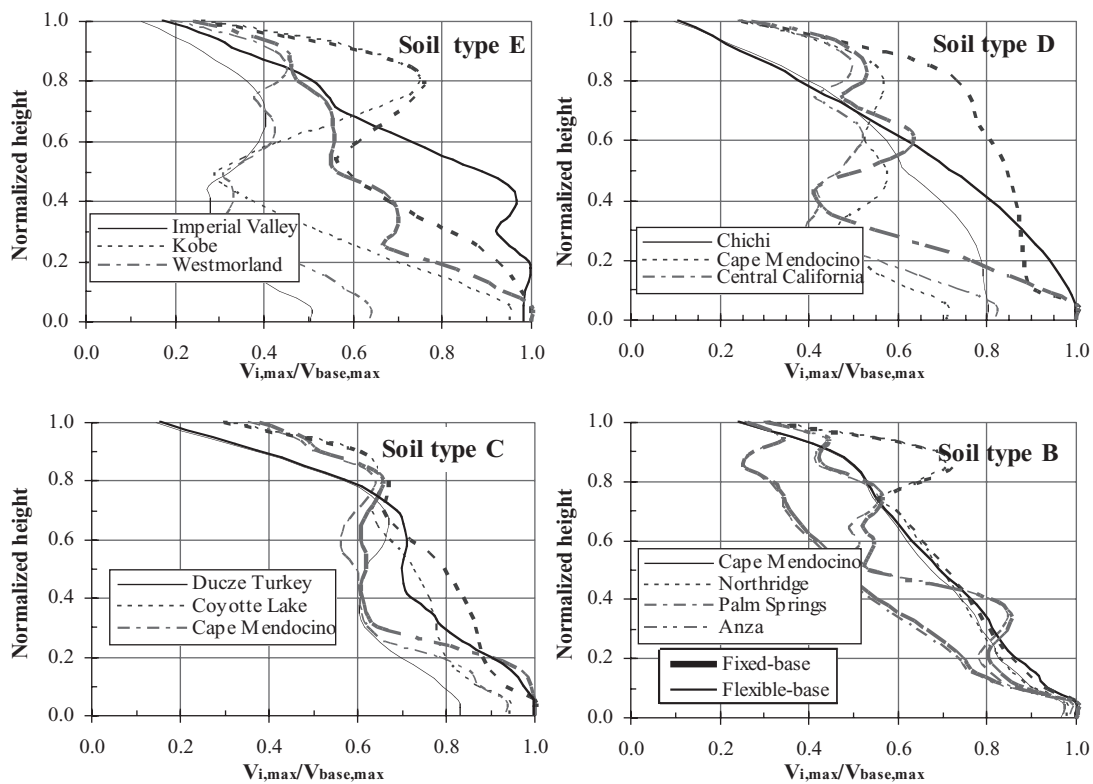


Figure 14. Distribution of maximum store shear-to-maximum base shear ratio over the normalized height of the 20-storey building for fixed-base and flexible-base models subjected to NFE records (scaled to match $S_{a,max}$ of IBC 2000) for soil types E, D, C, and B

case, other alternative measures, e.g., by considering several design ground motions, might need to be considered.

6.4 Analysis of 6-storey building

The response of the fixed-base and flexible-base models (for the four types of soil) of the 6-storey building when subjected to the same set of NFE records is evaluated. Table 8 shows the influence of scaling criteria of the NFE record on the SSI effects of the 6-storey frame for soil types E, D, C and B.

From the table, it can be seen that effects of SSI vary significantly according to the characteristics of the NFE record, the scaling criterion and the indicator representing the SSI, which confirms the previous findings drawn from the response of the 20-storey building.

7. CONCLUSIONS AND RECOMMENDATIONS

The nonlinear performance of two-dimensional 20-storey and 6-storey reinforced concrete (RC) building frames subjected to a set of 13 near-fault earthquake (NFE) ground motions having different characteristics have been presented. The NFE ground motions were selected on the basis of distance

Table 7. Influence of scaling criteria of NFE records on the SSI represented by the ratios of the maximum inter-storey drift and base shear of the flexible- and fixed-base models of the 20-storey building for soil types E, D, C and B

Soil type	Earthquake	Scaling criteria					
		$S_{a,max} = S_{a,max}$ of IBC		$S_a(T_{fixed}) = S_a(T_{fixed})$ of IBC		$S_a(T_{flexible}) = S_a(T_{flexible})$ of IBC	
		ID _{SSI} ratio ^a	V _{SSI} ratio ^b	ID _{SSI} ratio	V _{SSI} ratio	ID _{SSI} ratio	V _{SSI} ratio
E	Imperial Valley	1.17	0.51	1.20	0.53	1.11	0.61
	Kobe	1.07	0.95	1.02	0.97	1.22	1.08
	Westmorland	1.06	0.64	0.87	0.76	1.25	0.94
D	Cape Mendocino	1.04	0.71	1.09	0.78	1.00	0.83
	Central California	0.90	0.82	0.98	0.91	1.05	1.01
	Chichi	1.61	0.80	1.75	0.84	2.12	0.96
C	Cape Mendocino	0.98	0.93	0.98	1.02	0.98	1.02
	Coyote Lake	1.02	0.94	1.01	0.96	1.01	0.97
	Duzce Turkey	1.11	0.83	1.15	0.81	1.13	0.82
B	Anza	0.99	0.99	1.00	1.00	1.00	1.00
	Cape Mendocino	1.02	0.97	1.03	0.98	1.02	0.98
	Northridge	0.99	0.98	1.00	0.99	1.00	0.99
	Palm Springs	0.99	0.97	0.99	0.98	0.99	0.97

$$^a \text{ID}_{\text{SSI}} \text{ ratio} = \frac{\text{ID}_{\text{max}} \text{ for Flexible-base model}}{\text{ID}_{\text{max}} \text{ for Fixed-base model}}.$$

$$^b \text{V}_{\text{SSI}} \text{ ratio} = \frac{\text{V}_{\text{base,max}} \text{ for Flexible-base model}}{\text{V}_{\text{base,max}} \text{ for Fixed-base model}}.$$

Table 8. Influence of scaling criteria of NFE records on the SSI represented by the ratios of the maximum inter-storey drift and base shear of the flexible- and fixed-base models of the 6-storey building for soil types E, D, C and B

Soil type	Earthquake	Scaling criteria					
		$S_{a,max} = S_{a,max}$ of IBC		$S_a(T_{fixed}) = S_a(T_{fixed})$ of IBC		$S_a(T_{flexible}) = S_a(T_{flexible})$ of IBC	
		ID _{SSI} ratio ^a	V _{SSI} ratio ^b	ID _{SSI} ratio	V _{SSI} ratio	ID _{SSI} ratio	V _{SSI} ratio
E	Imperial Valley	1.87	0.75	1.89	0.79	1.81	0.89
	Kobe	1.39	0.80	1.17	0.97	1.02	0.99
	Westmorland	1.02	0.42	0.83	0.48	1.58	0.88
D	Cape Mendocino	1.37	0.80	1.48	0.72	1.36	0.95
	Central California	1.10	0.65	1.13	0.88	1.42	0.90
	Chichi	1.14	0.96	1.13	0.98	1.06	0.99
C	Cape Mendocino	1.01	0.96	1.00	1.00	1.01	1.00
	Coyote Lake	1.00	0.98	1.00	1.00	1.00	0.99
	Duzce Turkey	1.02	0.94	1.02	0.97	1.02	0.94
B	Anza	1.00	0.99	1.01	1.01	1.00	1.01
	Cape Mendocino	1.00	0.99	1.01	1.00	1.00	1.00
	Northridge	1.01	1.00	1.01	1.00	1.01	1.00
	Palm Springs	0.99	1.00	0.99	1.00	0.99	1.00

$$^a \text{ID}_{\text{SSI}} \text{ ratio} = \frac{\text{ID}_{\text{max}} \text{ for Flexible-base model}}{\text{ID}_{\text{max}} \text{ for Fixed-base model}}.$$

$$^b \text{V}_{\text{SSI}} \text{ ratio} = \frac{\text{V}_{\text{base,max}} \text{ for Flexible-base model}}{\text{V}_{\text{base,max}} \text{ for Fixed-base model}}.$$

from the site and the soil type characterized by the shear wave velocity. The seismic performance of the structures was evaluated using the roof displacement time history, inter-storey drift, and shear distribution along the height of the building. To study the effect of soil conditions on the seismic performance of the buildings, the response of the building with flexible-base models is compared to that of a fixed-base model. The characteristics of the flexible-base models cover four types of soils, namely, soft soil, medium soil, stiff soil and a rock soil, as classified by the International Building Code (IBC). In the nonlinear analysis, three criteria for scaling the records were considered, namely, same maximum spectral acceleration, same spectral acceleration at fundamental period of fixed-base model, and same spectral acceleration at fundamental period of flexible-base model. The soil–structure effects (SSI) for each scaling criteria is evaluated using two indicators, namely ID_{SSI} and V_{SSI} , that are defined as the ratios of the maximum inter-storey drift and base shear of the flexible- and fixed-base models, respectively.

Based on the conducted analysis, the following conclusions and recommendations are made:

- (1) SSI effects could vary significantly according to the characteristics of the NFE record, the scaling criterion and the indicator representing the SSI. This observation is valid for tall and low-rise structures.
- (2) Similar to the response of structures to far-field ground motions, the effect of SSI on the seismic performance of structures subjected to NFE ground motions is more pronounced in soft and medium soil types, and has less and negligible effects in stiff and rock soil types, respectively.
- (3) The increase in inter-storey drift (ID) due to SSI for some NFE records is characterized by a more uniform ID along the height with higher ID values at the lower-level storeys when compared to the fixed-base model. This can be attributed to the lumped rotation at the lower floor of the building.
- (4) In order to evaluate the seismic performance of structures on flexible soil in the near-fault region, it is recommended to conduct a site study to determine a design ground motion record or spectrum for the specific site soil conditions. On the other hand, attention should be paid to whether considering SSI is beneficial or detrimental to the seismic performance of the structure when subjected to a particular ground motion. Consequently, in the former case, other alternative measures, e.g. by considering several design ground motions, might need to be considered.

It should be noted that the above conclusions are based on limited analysis of specific structures and specific NFE records combinations. More analysis covering more cases will need to be conducted to refine or generalize the conclusions.

REFERENCES

- Alavi B, Krawinkler H. 2004. Behavior of moment resisting frame structures subjected to near-fault ground motions. *Earthquake Engineering and Structural Dynamics* **33**(6): 687–706.
- ATC-40. 1996. Seismic evaluation and retrofit of concrete buildings. *Report No. SSC 96-01*. Applied Technology Council, California Seismic Safety Commission: Redwood City, CA.
- Attalla MR, Paret TF, Freeman SA. 1998. Near-source behavior of buildings under pulse-type earthquakes. In *Proceedings, Sixth US National Conference on Earthquake Engineering*, Earthquake Engineering Research Center: Oakland, CA.
- Avilés J, Pérez-Rocha LE. 2005. Design concepts for yielding structures on flexible foundation. *Engineering Structures* **27**(3): 443–454.
- Avilés J, Suárez M. 2002. Effective periods and dampings of building-foundation systems including seismic wave effects. *Engineering Structures* **24**(5): 553–562.
- Avilés J, Suárez M, Sánchez-Sesma FJ. 2002. Effects of wave passage on the relevant dynamic properties of structures with flexible foundation. *Earthquake Engineering and Structural Dynamics* **31**(1): 139–159.
- CSA. 1994. *Design of Concrete Structures*. Standard A23.3-94. Canadian Standards Association: Rexdale, Ontario.

- Dutta SC, Bhattacharya K, Roy R. 2004. Response of low-rise buildings under seismic ground excitation incorporating soil-structure interaction. *Soil Dynamics and Earthquake Engineering* **24**(12): 893–914.
- El-Sheikh AMH. 2002. Response of RC structures to near fault earthquake records. Master's thesis, McMaster University, Ontario.
- FEMA 273/274. 1997. *NEHRP Guidelines for the Seismic Rehabilitation for Buildings*. Federal Emergency Management Agency: Washington, DC.
- FEMA 440. 2005. Improvement of nonlinear static seismic analysis procedures. *Report No. FEMA 440*, Federal Emergency Management Agency, prepared by Applied Technology Council. <http://www.atcouncil.org/pdfs/FEMA440CDdraftcameraready.pdf> [3 February 2006].
- Gazetas G. 1991. Formulas and charts for impedances of surface and embedded foundations. *Journal of Geotechnical Engineering, ASCE* **117**(9): 1363–1381.
- Halabian AM, El-Naggar MH. 2002. Effect of non-linear soil-structure interaction on seismic response of tall slender structures. *Soil Dynamics and Earthquake Engineering* **22**(8): 639–658.
- IBC. 2000. *International Building Code*. International Code Council: Falls Church, VA.
- Iwan WD, Huang CT, Guyader AC. 2000. Important features of the response of inelastic structures to near-field ground motion. In *Proceedings, Twelfth World Conference on Earthquake Engineering*, Paper No. 1740, New Zealand Society for Earthquake Engineering: New Zealand.
- Kobayashi H, Seo K., Midorikawa S. 1986. *Estimated Strong Ground Motions in the Mexico City Earthquake: The Mexico Earthquakes 1985, Factors Involved and Lessons Learned*. American Society of Civil Engineers: New York.
- Krawinkler H, Medina R, Alavi B. 2003. Seismic drift and ductility demands and their dependence on ground motions. *Engineering Structures* **25**(5): 637–653.
- Li K. 1999. *CANNY99: 3-Dimensional Nonlinear Static and Dynamic Structural Analysis, Computer Program, User's Manual*. CANNY Structural Analysis: Vancouver, BC, Canada.
- Liao WI, Loh CH, Wan S. 2001. Earthquake responses of RC moment frames subjected to near fault ground motions. *Structural Design of Tall Buildings* **10**: 219–229.
- Makris N, Roussos Y. 1998. Rocking response and overturning of equipment under horizontal pulse-type motions. *Report PEER-1998/05*. University of California: Berkeley, CA.
- Malhotra PK. 1999. Response of buildings to near-field pulse like ground motions. *Earthquake Engineering and Structural Dynamics* **28**(11): 1309–1326.
- Mylonakis G, Gazetas G. 2000. Seismic soil–structure interaction: beneficial or detrimental?. *Journal of earthquake Engineering* **4**(3): 277–301.
- NBCC. 1995. *National Building Code of Canada*. National Research Council of Canada: Ottawa, Ontario.
- PEER. 2005. Strong motion database. <http://peer.berkeley.edu/smcat/index.html> [December 2005].
- Singh JP. 1995. Characterization of ground motion for severity and damage potential. NISEE, University of California, Berkeley. <http://nisee.berkeley.edu/lessons/singh.html> [November 2005].
- Suzuki S, Asano K. 2000. Simulation of near source ground motion and its characteristics. *Soil Dynamics and Earthquake Engineering* **20**(1–4): 125–136.
- Vision 2000 Committee. 1995. *Performance Based Seismic Engineering of Buildings*. Structural Engineers Association of California (SEAOC): Sacramento, CA.
- Wolf JP. 1985. *Dynamic Soil–Structure Interaction*. Prentice-Hall: Englewood Cliffs, NJ.
- Wolf JP, Deeks AJ. 2004. *Foundation Vibration Analysis: A Strength-of-Materials Approach*. Elsevier: Oxford, UK.