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Soil-structure interaction vs Site effect for seismic design of tall buildings on soft soil

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Abstract. In this study, in order to evaluate adequacy of considering local site effect, excluding soil-structure interaction (SSI) effects in inelastic dynamic analysis and design of mid-rise moment resisting building frames, three structural models including 5, 10, and 15 storey buildings are simulated in conjunction with two soil types with the shear wave velocities less than 600 m/s, representing soil classes D_e and E_e according to the classification of AS1170.4-2007 (Earthquake actions in Australia) having 30 m bedrock depth. Structural sections of the selected frames were designed according to AS3600:2009 (Australian Standard for Concrete Structures) after undertaking inelastic dynamic analysis under the influence of four different earthquake ground motions. Then the above mentioned frames were analysed under three different boundary conditions: (i) fixed base under direct influence of earthquake records; (ii) fixed base considering local site effect modifying the earthquake record only; and (iii) flexible-base (considering full soil-structure interaction). The results of the analyses in terms of base shears and structural drifts for the above mentioned boundary conditions are compared and discussed. It is concluded that the conventional inelastic design procedure by only including the local site effect excluding SSI cannot adequately guarantee the structural safety for mid-rise moment resisting buildings higher than 5 storeys resting on soft soil deposits.

Keywords: soil-structure interaction; local site effect; inelastic dynamic analysis; mid-rise moment resisting building frames

1. Introduction

Nowadays, the new and emerging concept of seismic structural design, the so-called performance-based design, requires careful consideration of all aspects involved in structural analysis. Performance-based engineering (PBE) is a technique for seismic evaluation and design using performance level prediction for safety and risk assessment. Over the past few years, application of performance-based seismic design concepts has been promoted and developed. The development of this approach has been a natural outgrowth of the evaluation and upgrade process for existing buildings. Performance objectives are expressed as an acceptable level of damage, typically categorised as one of several performance levels. Performance levels describe the state of

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structures after being subjected to a certain hazard level and are classified as: *fully operational*, *operational*, *life safe*, *near collapse*, or *collapse* (Vision 2000, 1995, FEMA 273 1997). Overall lateral deflection, ductility demand, and inter-storey drifts are the most commonly used damage parameters. The above mentioned five qualitative performance levels are related to the corresponding quantitative maximum inter-storey drifts of: 0·2%, 0·5%, 1·5%, 2·5%, and > 2·5%, respectively.

The seismic excitation experienced by a structure is a function of local site effect and dynamic soil-structure interaction (SSI) influences. The treatment of soil-structure interaction effects in the analysis of structures founded on the surface or embedded in the soil is still one of the most discussed and challenging issues in the field of seismic design and requalification of different structures. Although building structures generally possess enough capacity reserves to sustain higher loads, the conservative design procedures have to be replaced by more realistic methods due to the vast requalification effort required for components and systems (Halbritter *et al*. 1998). Wave propagation theory denotes that soil layers modify the attributes of the input seismic waves while passing though the soil layers according to Kobayashi *et al*. (1986). The amplitude and frequency content of seismic shear waves reaching the earth's surface is dependent on site soil conditions. Soil amplification increases the ground motion intensity due to the dynamic response of local soil layers (Seed and Idriss 1969). This phenomenon is called “Local Site Effect”.

It is well established that local site conditions and near surface topography can exert a crucial influence on the severity of building damage and its spatial distribution during earthquakes. According to Adam *et al*. (2004), engineers have traditionally evaluated such influence using simple models based on one-dimensional description of local soil profile and seismic wave propagation with reasonable success. However, recent events such as the 1995 Hyogoken Nanbu (Japan) earthquake with its narrow “intensified damage” belt crossing the city of Kobe and causing the death of over 6000 people, have disclosed a remarkable complexity in seismic amplification patterns due to unfavourable combinations of seismic source and near surface geology.

During the last 20 years, a large number of observational studies have striven to evaluate the importance of the different factors involved in site response of soft soils concentrated in the determination of the amplitude of the transfer function relating input motion to the ground motion on top of the soft soils. In contrast, numerical evaluations of site effect have proceeded much farther, while the initial studies of site response in 2D homogeneous valleys were presented almost 20 years ago (Borcherdt 1994). Many researchers (e.g., Olsen *et al*. 1995, Furumura and Kennett 1998, Hokmabadi *et al*. 2014) focus on 3D numerical models, including the source and very complex soil structures. Such studies have been essential in providing valuable insights, and in building damage scenarios for specific cities. Nevertheless, they are clearly inadequate to address the more general issues that are at the core of building codes. Provisions have been developed in major codes of practices including the International Building Code (IBC 2012) to address this phenomenon. Soil amplification is also controlled by other parameters including the shear wave velocity gradient, thickness of soft soil layers, overall soil depth, impedance contrast at the soil-rock interface, and response spectrum representing the frequency content of seismic waves measured on the bedrock.

Local site effect in seismic analysis and design of structures is widely deemed to be adequate by practical engineers and major seismic codes around the globe to simulate complicated nature of seismic wave alteration and interaction with the soil and the structure. However, effects of dynamic soil-structure interaction (SSI), playing a very important and dominating role in the seismic design, have been undermined by most seismic codes and professionals. In this study, the adequacy of considering local site effect excluding soil-structure interaction effects in inelastic dynamic analysis and design of mid-rise moment resisting building frames resting on relatively soft soils are investigated.

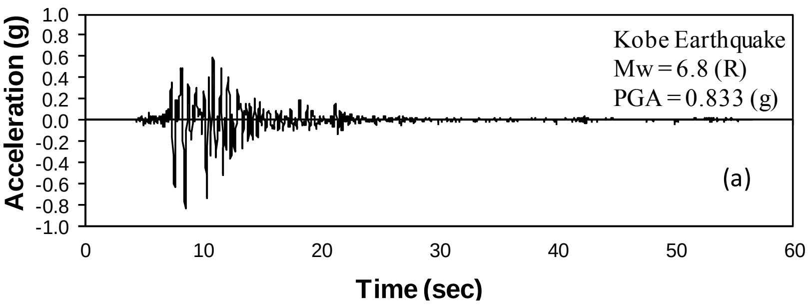
2. Nonlinear time-history dynamic analysis

Nonlinear time-history dynamic analysis is carried out in this study in order to determine dynamic response of the structural models. Time-history analysis is a step-by-step analysis of the dynamic response of a structure to a specified time dependant loading. The dynamic equilibrium equations to be solved can be presented as

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = r(t) \tag{1}$$

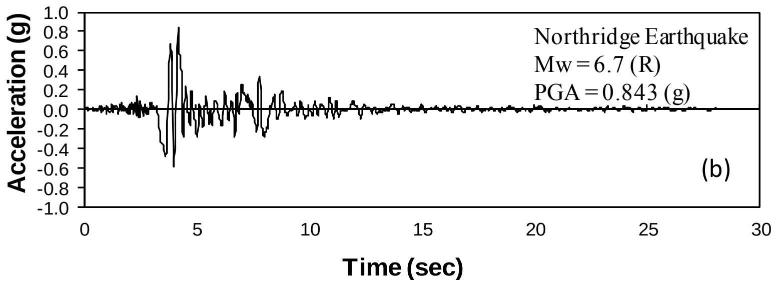
where, M, C, and K are the mass, damping and stiffness matrices, respectively; $u(t)$, $\dot{u}(t)$, and $\ddot{u}(t)$ are the displacements, velocities and accelerations of the structure, respectively, and $r(t)$ is the applied load to the structure. In nonlinear time-history analysis, the stiffness, damping, and load all depend upon the displacements, velocities, and time. This requires an iterative solution to the equations of motion. The nonlinear analysis internally solves the equations of motion at each output time step and at each load function time step, just as for linear analysis. In addition, a maximum sub step size smaller than the output time step is specified in order to reduce the amount of nonlinear iteration. In addition, the non-linear properties of the structure are considered as part of a time domain analysis. This approach is the most rigorous, and is required by some building codes for the appropriate design (e.g., ATC-40 1996, BSSC 2003).

In order to perform a comprehensive investigation on the seismic response of the structure models, two near field earthquake acceleration records including Kobe, 1995 (Fig. 1(a)) and Northridge, 1994 (Fig. 1(b)) and two far field earthquake acceleration records comprising El-Centro, 1940 (Fig. 1(c)) and Hachinohe, 1968 (Fig. 1(d)) are selected and utilised in time-history analysis. These earthquakes have been chosen by the International Association for Structural Control and Monitoring for benchmark seismic studies (Karamodin and Kazemi 2008). The characteristics of the earthquake ground motions are summarised in Table 1.



(a)

Fig. 1 Earthquake records adopted in this study



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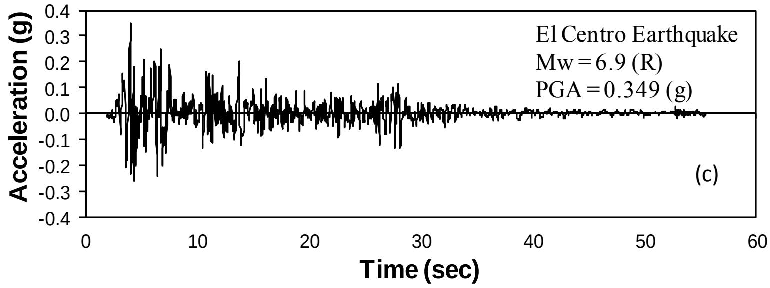

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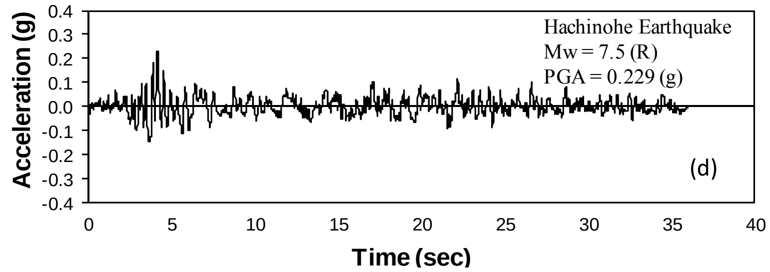


(d)

Fig. 1 Continued

Table 1 Earthquake ground motions used in this study

Earthquake	Country	Year	PGA (g)	*Mw* (R)	*T* (S) duration	Type	Hypocentral distance (km)	Reference
Northridge	USA	1994	0.843	6.7	30.0	Near field	9.2	PEER (2012)
Kobe	Japan	1995	0.833	6.8	56.0	Near field	7.4	PEER (2012)
El Centro	USA	1940	0.349	6.9	56.5	Far field	15.69	PEER (2012)
Hachinohe	Japan	1968	0.229	7.5	36.0	Far field	14.1	PEER (2012)

According to Kramer (1996), relative lateral structural displacements of a soil-structure system consist of rocking component and distortion component. A simple analysis is sufficient to illustrate the most important effects of soil-structure interaction on the above mentioned components. Wolf (1985) considered a case of simple SDOF (single Degree of Freedom) oscillator, mounted on a rigid foundation to analytically investigate the effects of SSI on the lateral displacement components. He concluded that the effects of SSI reduce the maximum structural distortions by an amount that increased with the stiffness ratio between the structure and the subsoil. However, those effects increase the rocking component and consequently amplify the overall lateral displacements by an amount that increased with stiffness ratio between the structure and the subsoil. In this study, in order to investigate the above mentioned effects, a Multi Degree of Freedom (MDOF) structural model is employed. Thus, inter-storey drifts can be determined and utilised for investigating the performance levels of the building structures under the influence of soil-structure interaction.

3. Studied building frames

According to Chandler *et al*. (2010), mid-rise buildings are aggregation of dwelling buildings ranging from 5 to 15 stories. With respect to this definition, in order to cover this range, three structural models consisting of 5, 10, and 15 storey models, representing conventional types of mid-rise reinforced concrete moment resisting building frames have been selected in this study as per specifications summarised in Table 2. The selected span width conforms to architectural norms and construction practices of the conventional buildings in mega cities.

For the structural concrete utilised in this analysis and design, specified compressive strength (f'_c) and mass density (ρ) are assumed to be 32 MPa and 2400 kg/m^3, respectively. The modulus of elasticity of concrete (E) was calculated according to Clause 3.1.2.a of AS3600 (2009) (Australian Standard for Concrete Structures) as follows

$$E = (\rho)^{1.5} \times (0.043\sqrt{f'_c}) \tag{2}$$

In this study, structural sections of the models are designed based on inelastic method assuming elastic-perfectly plastic behaviour for the structural members. For this purpose, structural members of the models (Table 2) are simulated in SAP2000 V14 software reflecting various geometries and properties of models S5 (5 storey), S10 (10 storey), and S15 (15 storey). Then gravity loads including permanent (dead) and imposed (live) actions are determined and applied to the structural models, in accordance with AS/NZS1170.1 (2002) (Permanent, imposed and other actions). The values of permanent and imposed actions are determined as uniform distributed loads over the

Table 2 Dimensional characteristics of the studied frames

Reference name (Code)	Number of stories	Number of bays	Story height (m)	Bay width (m)	Total height (m)	Total width (m)	Spacing of the frames into the page (m)
S5	5	3	3	4	15	12	4
S10	10	3	3	4	30	12	4
S15	15	3	3	4	45	12	4

floors according to AS/NZS1170.1 (2002), considering the spacing of the frames being 4 m and permanent action (G) equal to 6 kPa and imposed action (Q) equal to 2 kPa.

Then, inelastic time-history dynamic analyses under the influence of four earthquake ground motions, shown in Table 1, are performed on the structural models. The generic process of inelastic analysis is similar to conventional elastic procedure. The primary difference is that the properties of the components of the model include plastic moment in addition to the initial elastic properties. These are normally based on approximations derived from test results on individual components or theoretical analyses (ATC-40 1996). In this study, inelastic bending is simulated in structural elements by specifying a limiting plastic moment. When the plastic moment is specified, the value may be calculated by considering a flexural structural member of width b and height h with yield stress σ_y. If the member is composed of a material that behaves in an elastic-perfectly plastic manner (Fig. 2), the plastic resisting moments (M^P) for rectangular sections can be computed as follows

$$M^P = \sigma_y \left(\frac{bh^2}{4} \right) \tag{3}$$

Present formulations adopted in this study for inelastic analysis and design assume that structural elements behave elastically until reaching the defined plastic moment. The section at which the plastic moment (M^P) is reached can continue to deform, without inducing additional resistance. In this study, plastic moments, (M^P), for each concrete section of models S5 (5 storey), S10 (10 storey), and S15 (15 storey) have been determined according to Equation (3) and assigned to the sections considering the yield stress of concrete material (σ_y) equal to the compressive strength of concrete (f'_c). In addition, geometric nonlinearity and P-Delta effects are considered according to AS3600 (2009) and cracked sections for the reinforced concrete sections are taken

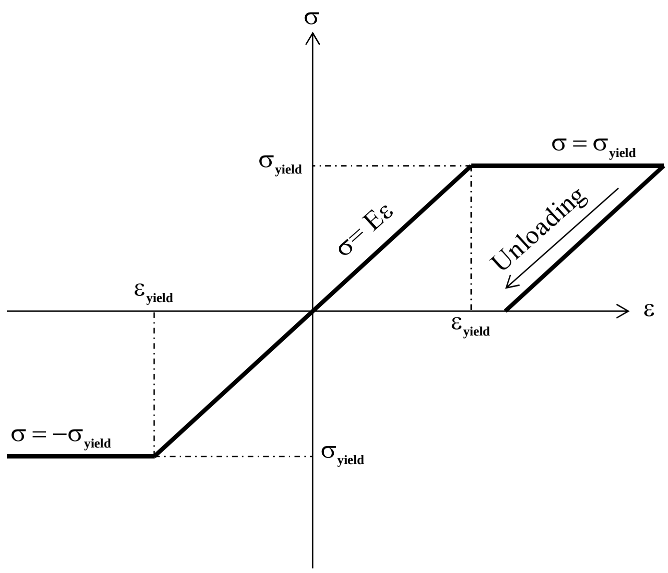


Fig. 2 Elastic-perfectly plastic behaviour of structural elements

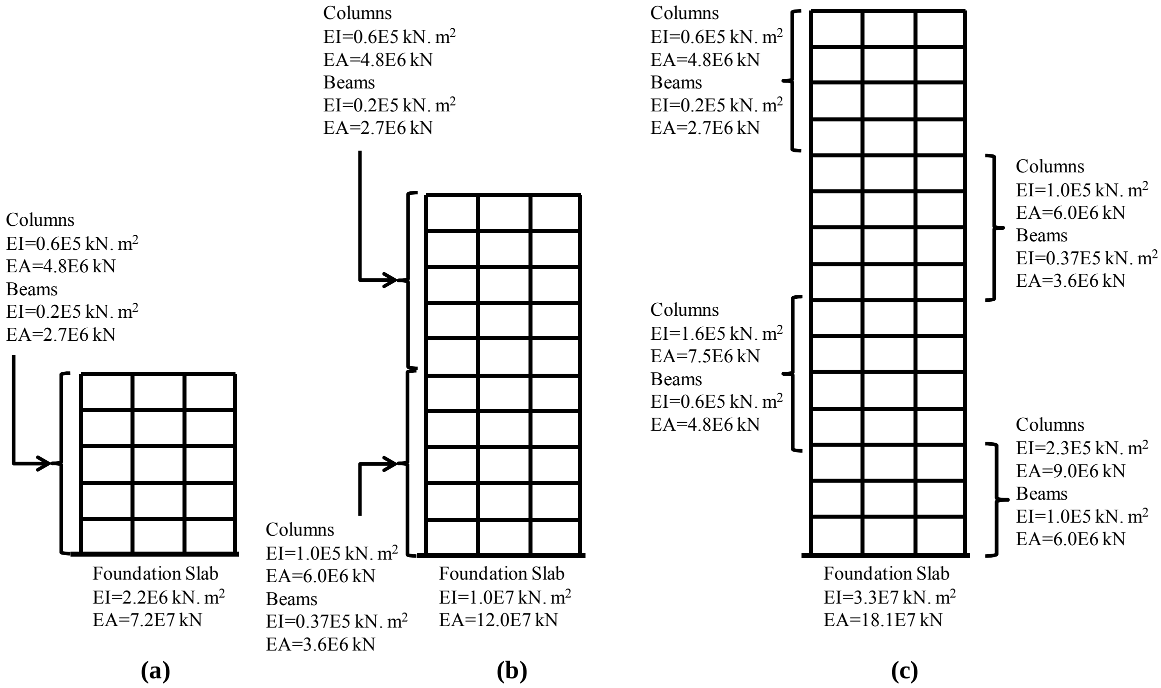


Fig. 3 Concrete sections designed for the adopted frames based on inelastic design method; (a) 5 storey model (S5); (b) 10 storey model (S10); (c) 15 storey model (S15)

into consideration by multiplying the cracked section coefficients by the stiffness values of the structural members (*EI*) according to ACI318 (2002). Based on this standard, cracked section coefficients are 0.35 and 0.70 for beams and columns, respectively.

Afterwards, structural members are designed in accordance with AS3600 (2009) (Australian Standard for Concrete Structures) in a way that performance levels of the designed models stay in *life safe* level by limiting the maximum inelastic inter-storey drifts to 1.5%. In order to determine inelastic inter-storey drifts for each two adjacent stories, maximum lateral deflections of each storey is derived from SAP2000 deflection history records. Using the maximum storey deflections, inelastic inter-storey drifts have been determined using the following equation based on AS 1170.4 (2007)

$$drift = (d_{i+1} - d_i)/h \tag{4}$$

where, d_{i+1} is deflection at (i + 1) level, d_i is deflection at (i) level, and h is the storey height.

In practical designs, it is often assumed that the storey deflection is equal to the horizontal displacement of the nodes on the level which may be due to translation, rotation, and distortion. In the final selection of the beam and column sections, constructability and norms have been considered. Fig. 3 summarises the concrete sections designed for the adopted frames based on inelastic structural design method.

4. Properties of utilized soils

According to available literature, generally when the shear wave velocity of the supporting soil is less than 600 m/s, the effects of soil-structure interaction on the seismic response of structural systems, particularly for moment resisting building frames, are significant (e.g., Veletsos and Meek 1974, Galal and Naimi 2008, Nateghi-A and Rezaei-Tabriz 2011, Tabatabaiefar *et al.* 2013a). Therefore, in this study, two relatively soft clayey soil samples with the shear wave velocity less that 600 m/s, representing soil classes D_e and E_e, according to AS 1170.4 (2007) have been utilised. Characteristics of the adopted soils are shown in Table 3. The subsoil properties have been extracted from actual in-situ and laboratory tests (Rahvar 2006a, b). Thus, these parameters have merits over the assumed parameters which may not be completely conforming to reality. The shear wave velocity values, shown in Table 3, have been obtained from down-hole test, which is a low strain in-situ test. This test generates a cyclic shear strain of about 10^{-4} percent where the resulting shear modulus is called $G_{\max}$. It should be noted that the shear wave velocity can be measured in the laboratory using bender element test (Fatahi *et al.* 2013). In the event of an earthquake, the cyclic shear strain amplitude increases and the shear strain modulus and damping ratio which both vary with the cyclic shear strain amplitude, change relatively. Damping and tangent module are selected to be appropriate to the level of excitation at each point in time and space which is called hysteretic damping algorithm. In this study, the tangent modulus function presented by Hardin and Drnevich (1972), known as Hardin model is employed in order to implement hysteretic damping to the numerical models. This model is defined as follows

$$M_s = \frac{1}{1 + \gamma / \gamma_{ref}} \tag{5}$$

where, M_s is the secant modulus ($G/G_{\max}$), γ is the cyclic shear strain, and γ_{ref} is Hardin/Drnevich constant. Bedrock depth is assumed to be 30 metres as the most amplification occurs within the first 30 metres of the soil profile, which is in agreement with most of modern seismic codes (e.g., ATC-40 1996, BSSC 2003). Those seismic codes evaluate local site effects just based on the properties of the top 30 meters of the soil profile. In addition, it is assumed that water table is below the bedrock level.

5. Numerical simulation

In this study, in order to investigate the inadequacy of considering local site effect in time history dynamic analysis of building frames excluding SSI effects, three structural models including 5, 10, and 15 storey buildings (Table 2) are simulated in conjunction with two soil types with the shear wave velocities less than 600m/s, representing soil classes D_e and E_e (Table 3) according to the classification of AS1170.4 (2007) (Earthquake actions in Australia) having 30 m bedrock depth. For numerical simulation of the models, FLAC2D has been used. FLAC2D (Fast Lagrangian Analysis of Continua) is a two-dimensional explicit finite difference program for engineering mechanics computations. This program can simulate behaviour of different types of structures and materials by elements which can be adjusted to fit the geometry of the model. Each element behaves according to a prescribed constitutive model in response to the applied forces or boundary restraints. The program offers a wide range of capabilities to solve complex problems in

Table 3 Geotechnical characteristics of the adopted soils in this study

Soil type (AS1170)	Shear wave velocity Vs (m/s)	Unified classification (USCS)	Maximum shear modulus G_{max} (kPa)	Poisson's ratio	Soil density ρ (kg/m^3)	c' (kPa)	ϕ' (degree)	Plasticity Index (PI)	Reference
D_e	320	CL	177,304	0.39	1730	20	19	20	Rahvar (2006a)
E_e	150	CL	33,100	0.40	1470	20	12	15	Rahvar (2006b)

mechanics such as inelastic analysis including plastic moment and simulation of hinges for structural systems.

In order to study the main differences between the local site effect and SSI influences on the seismic behaviour of the selected building frames, inelastic dynamic time history analyses are carried out, using FLAC2D software for the below mentioned three different cases.

Case 1

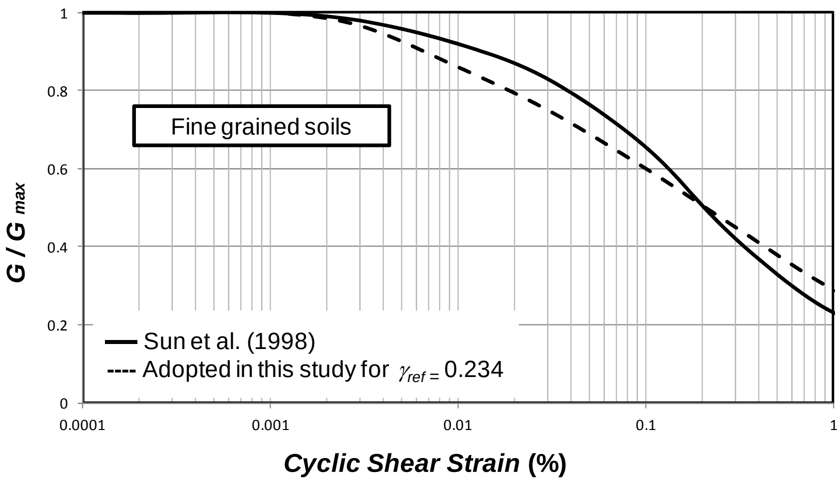
Fixed base columns on rigid ground under direct influence of the earthquake acceleration records summarised in Table 1 including Kobe, 1995 (Fig. 1(a)) and Northridge, 1994 (Fig. 1(b)), El-Centro, 1940 (Fig. 1(c)), and Hachinohe, 1968 (Fig. 1(d)). Fig. 5(a) illustrates an example of Case 1 for a fixed base 15 storey model under the direct influence of Northridge (1994).

Case 2

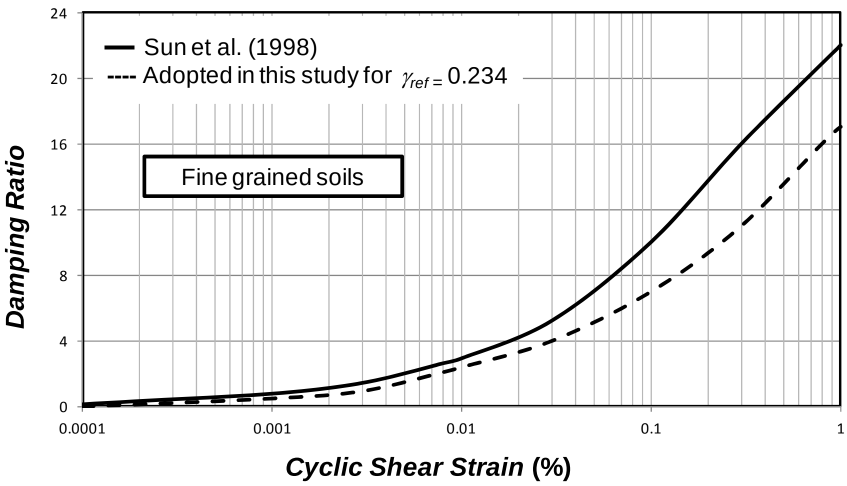
Fixed base columns on rigid ground considering local site effect modifying the earthquake records. To achieve this goal, four earthquake records summarised in Table 1 including Kobe, 1995 (Fig. 1(a)) and Northridge, 1994 (Fig. 1(b)), El-Centro, 1940 (Fig. 1(c)), and Hachinohe, 1968 (Fig. 1(d)) have been passed through 30 metres of soil classes D_e and E_e, respectively, using FLAC 2D software. After passing the earthquake acceleration records through 30 metres of soil classes D_e and E_e, it is observed that PGA (Peak Ground Acceleration) of the earthquake ground motions have been amplified by maximum 75% and 155% at the surface levels of soil classes D_e and E_e, respectively. The amplified PGA of the four earthquakes at the surface levels of soil classes D_e and E_e are summarised in Table 4. As mentioned earlier, Hardin model has been

Table 4 PGA of the used earthquake ground motions in this study at different levels

Earthquake	PGA (g) Bedrock level	PGA (g) Surface level soil D_e	PGA (g) Surface level soil E_e
Northridge	0.843	1.34	2.08
Kobe	0.833	1.45	2.15
El Centro	0.349	0.468	0.745
Hachinohe	0.229	0.398	0.503



(a)



(b)

Fig. 4 Adopted fitting curves for clay in this study; (a) Relations between G/G_{max} versus cyclic shear strain; (b) Relations between material damping ratio versus cyclic shear strain

employed in order to implement hysteretic damping to the numerical soil models. Adopted model in FLAC2D generates backbone curves represented by Sun *et al.* (1998) for fine grained soils**,** adopting γ_{ref} = 0.234 (Fig. 4) as numerical fitting parameter. Then, the resulting acceleration records at the ground surface have been recorded. In order to consider site effect, in the inelastic dynamic time history analysis, those resulting acceleration records at the ground surface were applied to the fixed base structure and the results of inelastic dynamic analyses were determined. Fig. 5(b) represents an example of Case 2 for a fixed base 15 storey model under the influence of the amplified record of Northridge (1994).

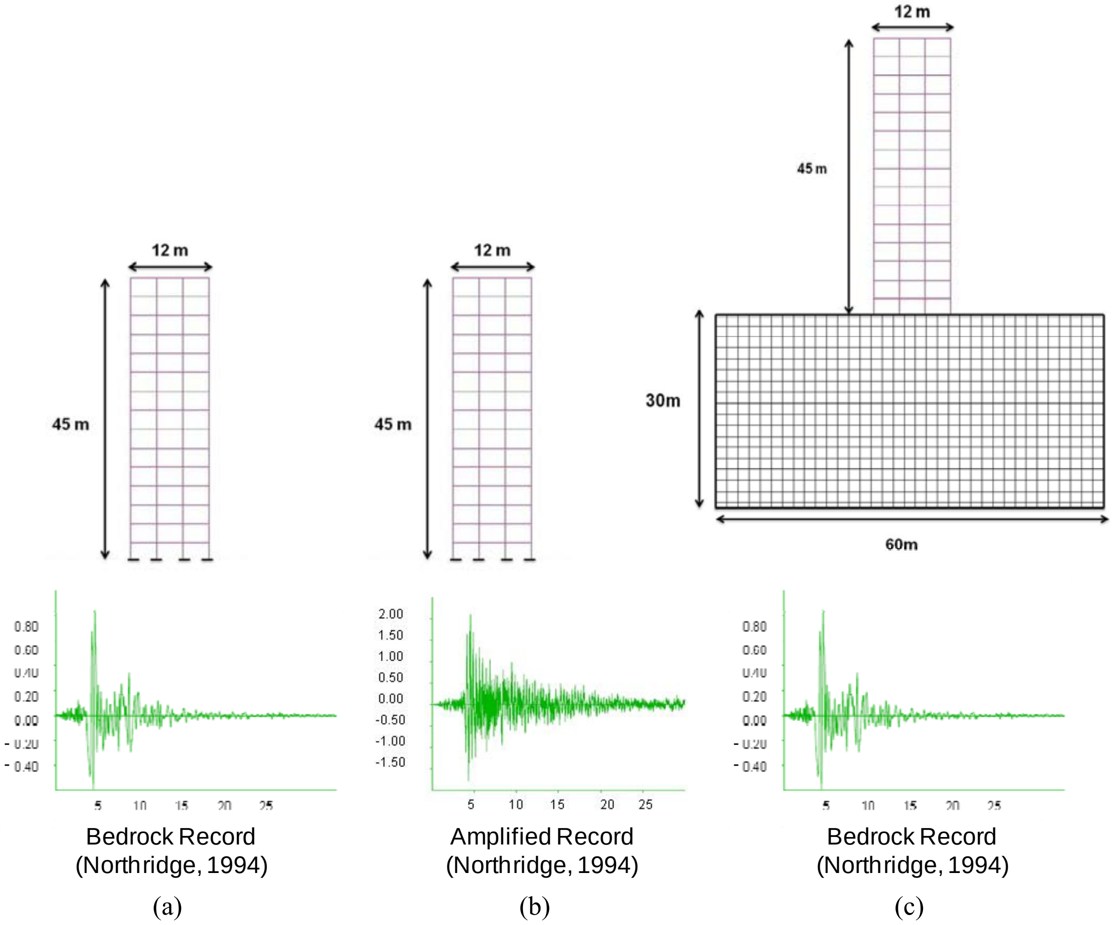


Fig. 5 Three different analysis cases employed in this study (a) fixed base under influence of bedrock record; (b) fixed base under influence of amplified record (Site effect); (c) flexible-base considering full soil-structure interaction

Case 3

Flexible base model considering soil medium underneath the structure which is called soil-structure model, employing direct method, to model and analyse dynamic soil-structure interaction. To model soil-structure system in direct method, a novel and enhanced soil-structure model is developed to simulate various aspects of complex dynamic soil-structure interaction in a realistic and rigorous manner. In direct method, the entire soil-structure system is modelled in a single step. The use of direct method requires a computer program that can treat the behaviour of both soil and structure with equal rigor simultaneously (Kramer 1996). Thus, finite difference software, FLAC2D V6.0, is utilised to model the soil-structure system and to solve the equations for the complex geometries and boundary conditions. The soil-structure model, shown in Fig. 6, employs beam structural elements to model beams, columns and the foundation slab. During analysis process, structural material could behave as an isotropic, linearly elastic material with no failure limit for elastic structural analysis or as an elastic-perfectly plastic material with a specified limiting plastic moment for inelastic structural analysis. Therefore, both elastic and plastic (inelastic) structural behaviour can be captured by the model in dynamic analysis. In addition, structural geometric nonlinearity (large displacements) has been accommodated in dynamic analysis. Two dimensional plane-strain grids composed of quadrilateral elements are utilised to model the soil medium. Nonlinear behaviour of the soil medium has been captured using backbone curves of shear modulus ratio versus shear strain (G/G_{max} - γ) and damping ratio versus shear strain (ξ - γ) adopting Mohr-Coulomb failure model. Employing the backbone curves for simulating nonlinear behaviour of the soil, in this study, fully nonlinear method for analysis of dynamic soil-structure interaction has been employed in order to attain rigorous and reliable results. Fully nonlinear method is capable to precisely model nonlinearity in dynamic analysis of soil-structure systems and follow any prescribed nonlinear constitutive relation (Fatahi and Tabatabaifar 2013, Tabatabaifar *et al*. 2013b). Similar to Case 2, in order to implement hysteretic damping to the numerical soil models, Hardin model, adopting γ_{ref} = 0.234 (Fig. 4) for clay as numerical fitting parameter, is employed.

The foundation facing zone in numerical simulations is separated from the adjacent soil zone by interface elements to simulate frictional contact. The interface between the foundation and soil is represented by normal (k_n) and shear (k_s) springs between two planes contacting each other and is modelled using linear spring system, with the interface shear strength defined by the Mohr–Coulomb failure criterion (Fig. 7). The relative interface movement is controlled by interface stiffness values in the normal and tangential directions. Normal and shear spring stiffness

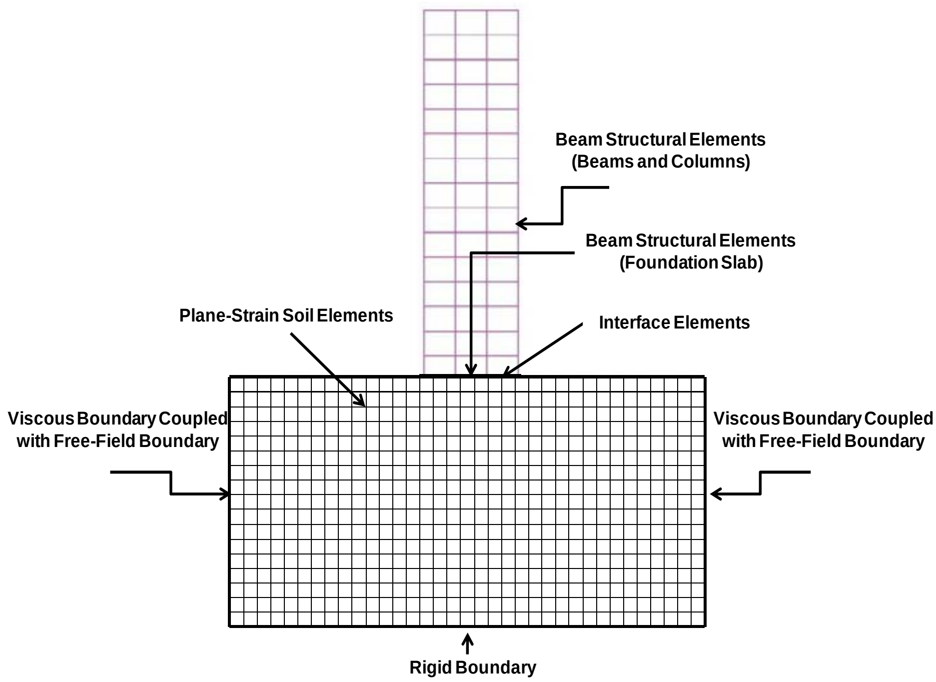


Fig. 6 Components of the soil-structure model

values for interface elements of the soil-structure model are set to ten times the equivalent stiffness of the neighbouring zone, based on recommended relationship by Rayhani and EL Naggar (2008) and Itasca Consulting Group (2008) for the isotropic soil medium, as follows

$$k_s = k_n = 10\left[\frac{(K + \frac{4}{3}G)}{\Delta z_{\min}}\right] \tag{6}$$

where, K and G are bulk and shear modulus of the neighbouring zone, respectively, and $\Delta z_{\min}$ is the smallest width of an adjoining zone in the normal direction. This is a simplifying assumption that has been only used for interface modelling. Since there is no large slip between the soil and

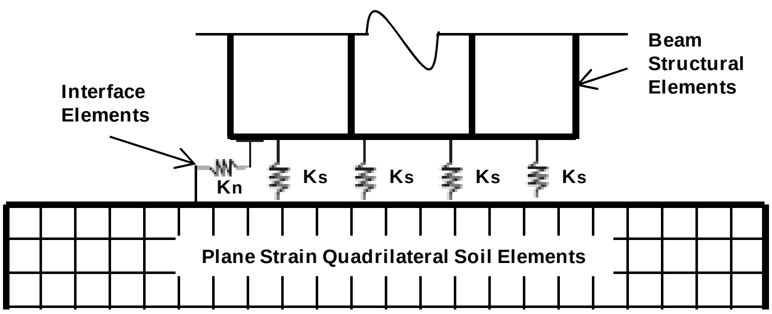


Fig. 7 Interface elements including normal and shear stiffness springs

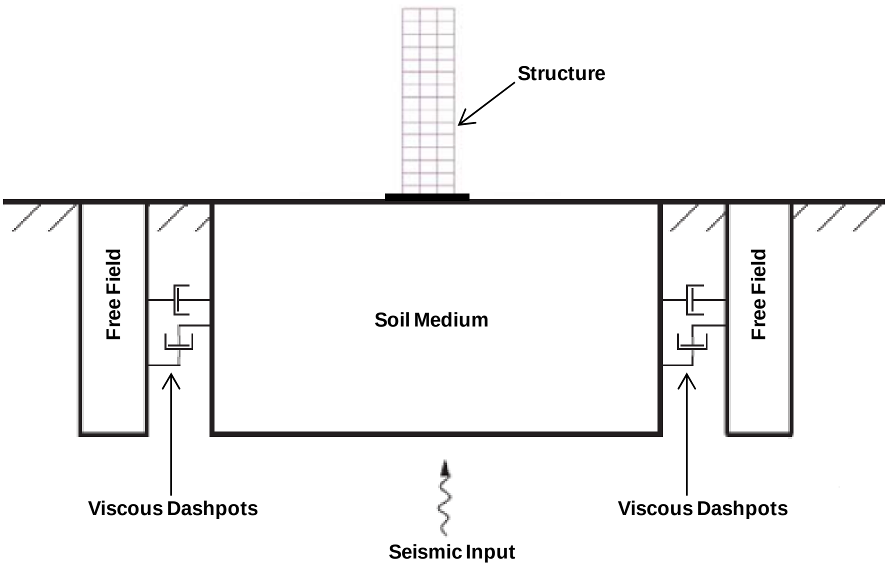


Fig. 8 Simulating lateral boundary conditions for soil-structure model

foundation in this study, this assumption does not influence the numerical results. In order to avoid reflection of outward propagating waves back into the model, quiet (viscous) boundaries comprising independent dashpots in the normal and shear directions are placed at the lateral boundaries of the soil medium. The lateral boundaries of the main grid are coupled to the free-field grids by viscous dashpots of quiet boundaries at the sides of the model to simulate the free-field motion which would exist in the absence of the structure (Fig. 8).

According to Rayhani and EL Naggar (2008), horizontal distance between soil boundaries is assumed to be five times the structural width (i.e., 60 m). Fully nonlinear method for dynamic analysis of soil-structure system, including variation of damping ratio and shear modulus reduction factor as mentioned above, has been used in order to model dynamic nonlinearity of soil material in a rigorous and reliable manner under the direct influence of the earthquake acceleration records summarised in Table 1 at the level of bedrock (Fig. 5(c)).

6. Results and discussions

The results of inelastic analyses in terms of base shears and inter-storey drifts under the influence of four mentioned earthquake ground motions are derived from FLAC2D history records and compared for the three mentioned cases. According to the base shear results, summarised in

Table 5 Base shear ratios of model S5 for three different cases

		Soil Type D_e				Soil Type Ee			
Earthquake	Case 1 Fixed-base model	Case 2 Fixed-base with site effect		Case 3 Flexible base		Case 2 Fixed-base with site effect		Case 3 Flexible base	
	V (kN)	V_{SE} (kN)	V_{SE} / V	$\tilde{V}$ (kN)	$\tilde{V}$ / V	V_{SE} (kN)	V_{SE} / V	$\tilde{V}$ (kN)	$\tilde{V}$ / V
Northridge	89	99	1.11	73	0.82	116	1.31	59	0.66
Kobe	130	152	1.17	106	0.81	178	1.37	85	0.65
El Centro	39	49	1.26	30	0.76	56	1.44	24	0.61
Hachinohe	47	57	1.22	36	0.76	67	1.42	29	0.61

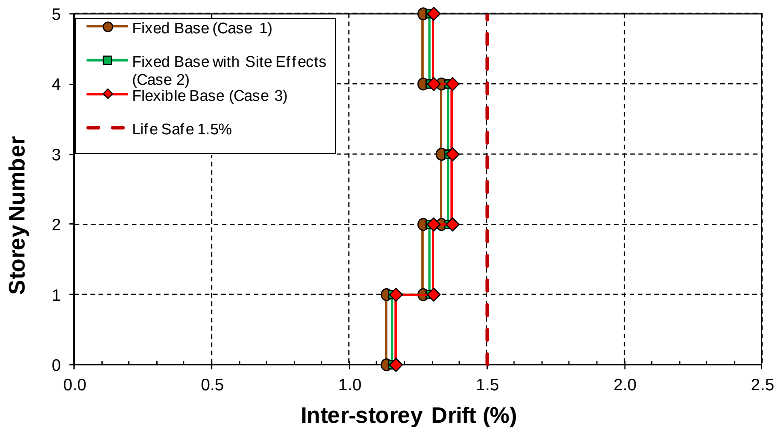
Table 6 Base shear ratios of model S10 for three different cases

		Soil Type D_e				Soil Type Ee			
Earthquake	Case 1 Fixed-base model	Case 2 Fixed-base with site effect		Case 3 Flexible base		Case 2 Fixed-base with site effect		Case 3 Flexible base	
	V (kN)	V_{SE} (kN)	V_{SE} / V	$\tilde{V}$ (kN)	$\tilde{V}$ / V	V_{SE} (kN)	V_{SE} / V	$\tilde{V}$ (kN)	$\tilde{V}$ / V
Northridge	289	358	1.24	214	0.74	456	1.58	147	0.51
Kobe	370	444	1.20	285	0.77	559	1.51	196	0.53
El Centro	132	181	1.37	90	0.68	218	1.65	53	0.40
Hachinohe	107	142	1.33	77	0.72	163	1.52	47	0.44

Table 7 Base shear ratios of model S15 for three different cases

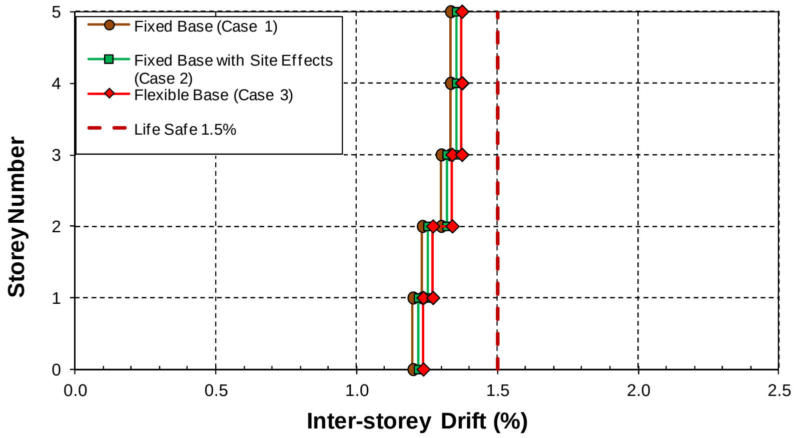
Earthquake	Case 1 Fixed-base model	Case 2 Fixed-base with site effect (Soil Type D_e)		Case 3 Flexible base (Soil Type D_e)		Case 2 Fixed-base with site effect (Soil Type Ee)		Case 3 Flexible base (Soil Type Ee)	
	V (kN)	V_{SE} (kN)	V_{SE} / V	$\tilde{V}$ (kN)	$\tilde{V}$ / V	V_{SE} (kN)	V_{SE} / V	$\tilde{V}$ (kN)	$\tilde{V}$ / V
Northridge	441	576	1.44	270	0.61	786	1.78	203	0.46
Kobe	550	770	1.40	352	0.64	940	1.71	264	0.48
El Centro	194	305	1.57	105	0.54	380	1.95	72	0.37
Hachinohe	167	255	1.53	94	0.57	304	1.82	63	0.38

Tables 5, 6, and 7 for models S5, S10, and S15 resting on soil classes D_e and E_e, respectively, it is observed that when local site effect in time history analysis is considered (Case 2), the base shears of the modelled structures are noticeably increases, by 11% in model S5 resting on soil class D_e and by 95% in model S15 resting on soil class E_e. Therefore, as a general trend while considering local site effect in time history analysis, by decreasing the shear wave velocity (V_s) of the subsoil or increasing the structural height, the base shear ratio of the models increase relatively. However, by incorporating full dynamic soil-structure interaction in the analysis (e.g., dynamic soil nonlinearity, material and geometric damping, and system natural period increment in Case 3) as a realistic simulation technique, the base shear in comparison to Case 1 significantly reduces (e.g., 18% reduction in model S5 resting on soil class D_e and 62% reduction in model S15 resting on soil class E_e). These results have good conformity to Section 5.6.2 of BSSC (2003) regulations as in this section reduction of base shear due to SSI is predicted. In general, by decreasing the

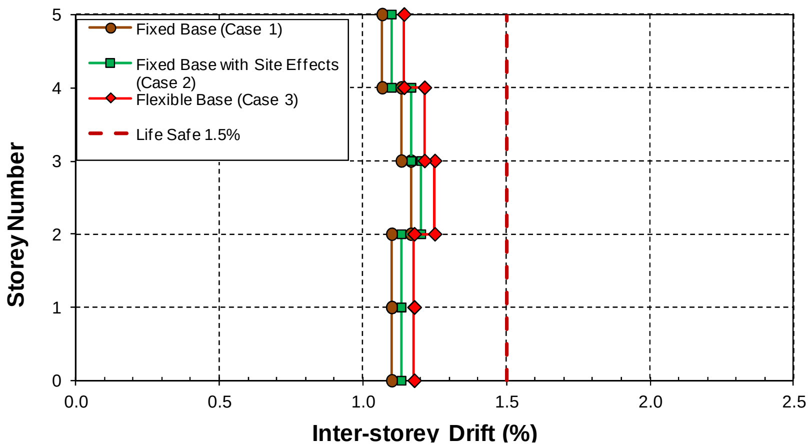


(a)

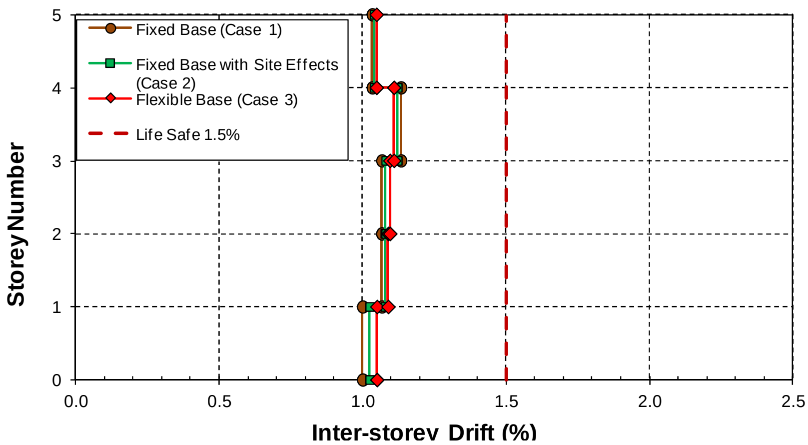
Fig. 9 Inter-storey drifts of model S5 resting on soil class D_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake



(b)



(c)

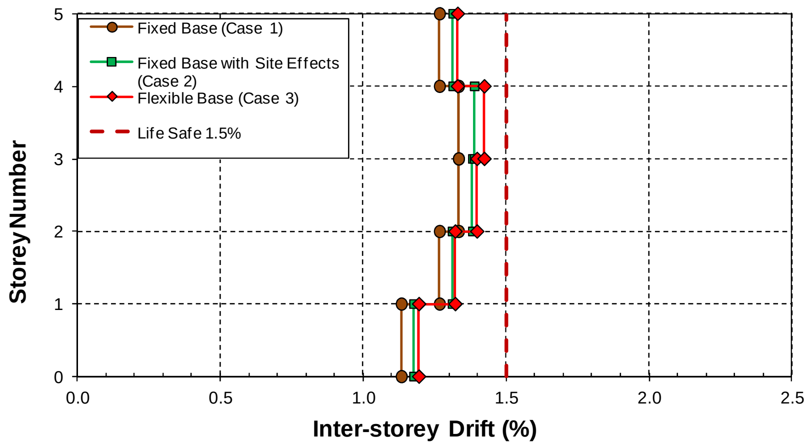


(d)

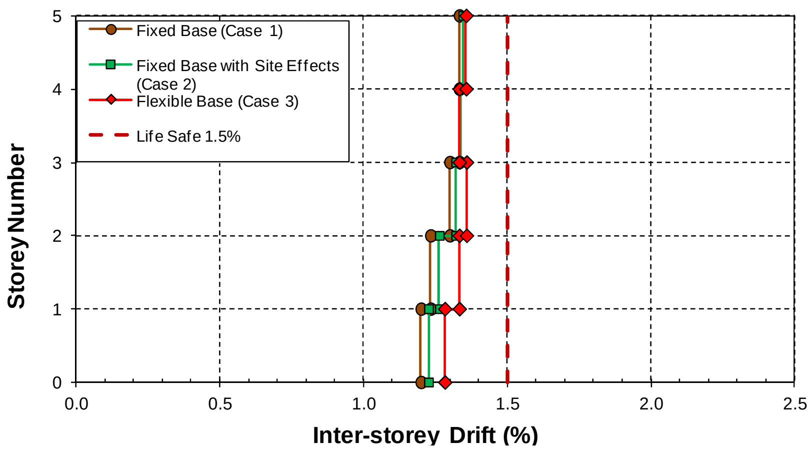
Fig. 9 Continued

shear wave velocity (V_s) or increasing the structural height, the base shear ratio of flexible base models decrease relatively. As a result, observing two totally different trends for Cases 2 and 3, it can be concluded that taking local site effect into account, excluding dynamic soil-structure interaction, results in unacceptably over conservative and unrealistic prediction of the base shear of the structure.

In order to investigate and compare the influence and importance of SSI and local site effect on the displacement response of the mid-rise moment resisting structures resting on relatively soft soils, predicted inter-storey drifts of models S5, S10, and S15 for the three cases are presented in Figs. 9 to 14. Based on the results of models S5, S10, and S15 resting on soil classes D_e (Figs. 9, 11 and 13), predicted inter-storey drifts of the models under the influence of local site effect

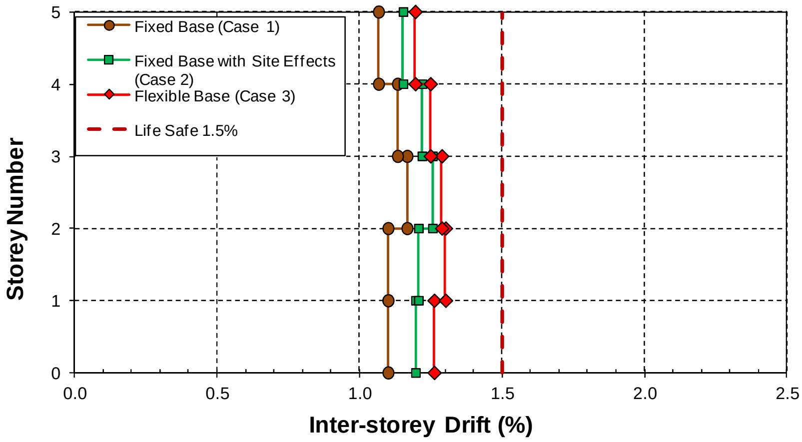


(a)

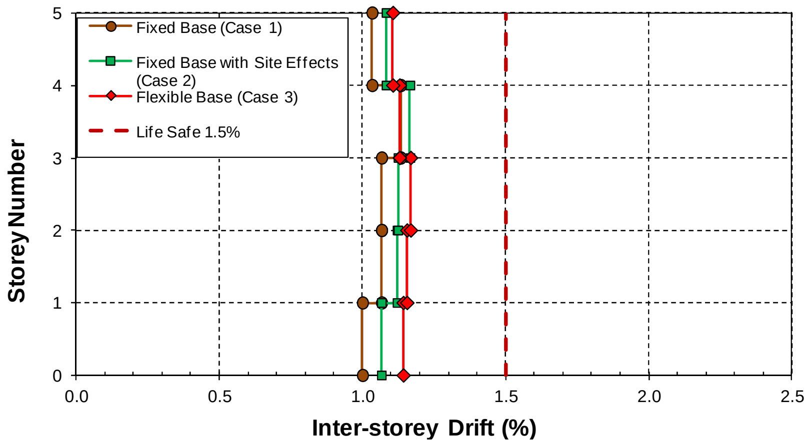


(b)

Fig. 10 Inter-storey drifts of model S5 resting on soil class E_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake



(c)

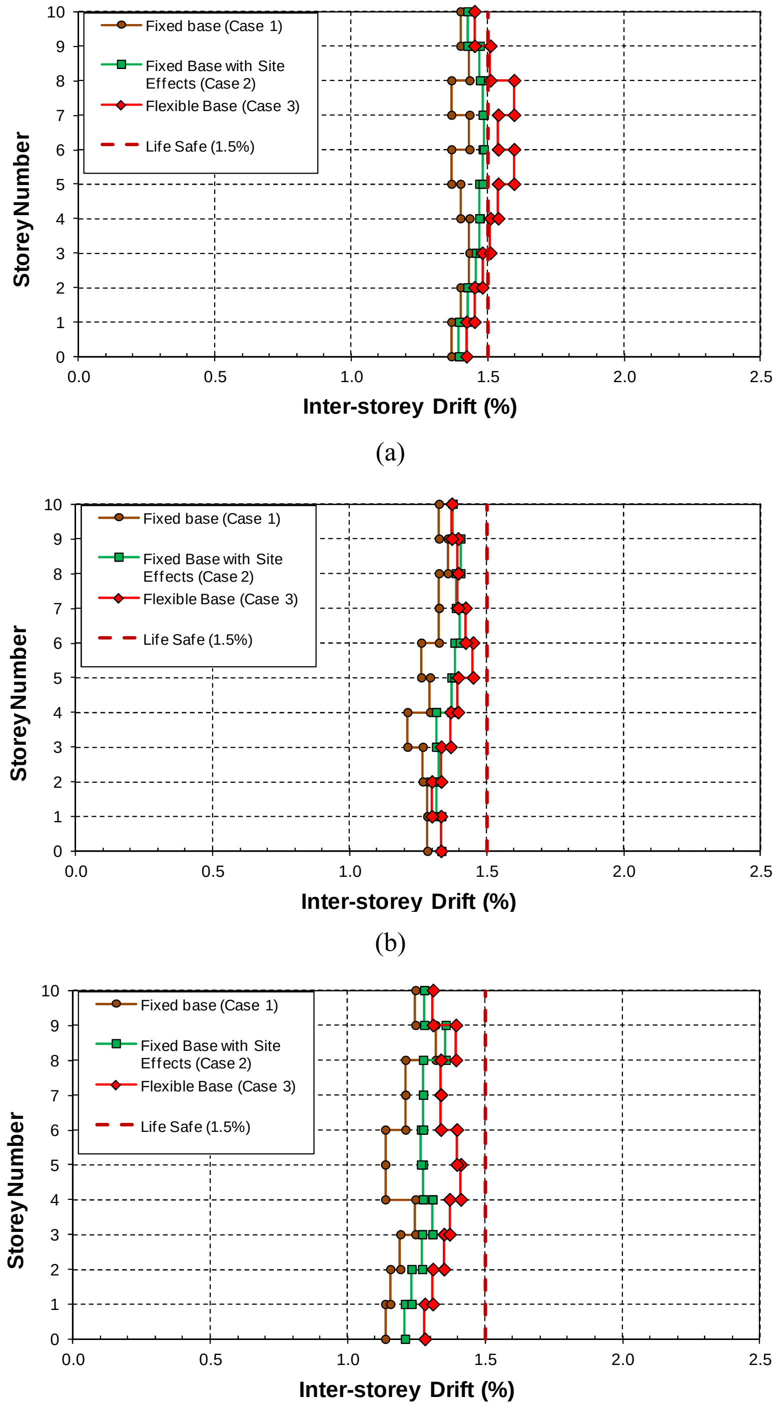


(d)

Fig. 10 Continued

(Case 2) are very close to the realistic results of SSI analyses predicted in Case 3 with the maximum difference being 6%. In addition, it is realised that the results of model S5 resting on soil class E_e (Fig. 10) indicate the same trend. Therefore, for the studied mid-rise structures resting on soil class D_e as well as 5 storey building frame resting on soil class E_e, by taking local site effect into account in the dynamic time history analyses almost realistic displacement response of the structures may be obtained without incorporating full dynamic soil-structure interaction in the analysis.

However, according to the inter-storey drift results of models S10 and S15 resting on soil class E_e (Figs. 12 and14), inter-storey drifts under the influence of local site effect (Case 2) are amplified in average by 11% and 23% in models S10 and S15, respectively, in comparison to Case 1 (fixed-base with no site effect). Nevertheless, they are considerably lower than the realistic results of SSI analyses predicted in Case 3, where by considering full soil-structure interaction, lateral

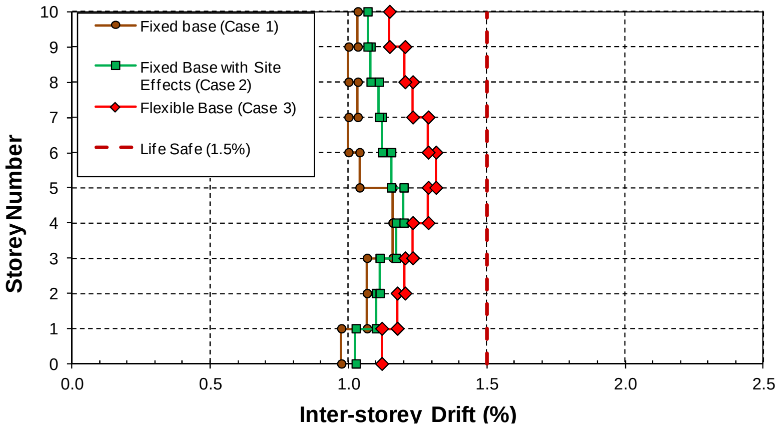


(a)

(b)

(c)

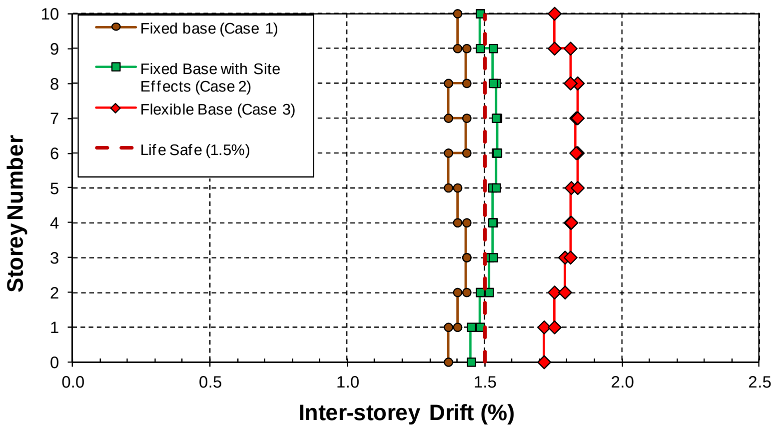
Fig. 11 Inter-storey drifts of model S10 resting on soil class D_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake



(d)

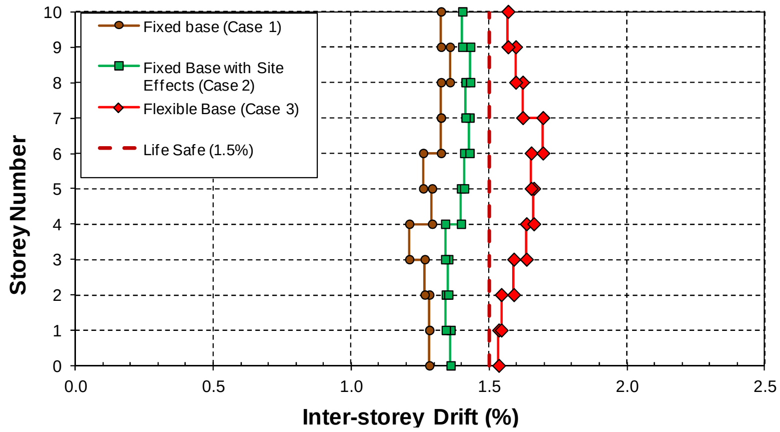
Fig. 11 Continued

deformations and inter-storey drifts are substantially amplified in average by 24% and 68% in models S10 and S15, respectively, in comparison to Case 1. In order to illustrate the influence of the site effect and SSI on the lateral deformations, lateral deflections of model S15 resting soil class E_e are shown in Fig. 15 as an example. Lateral deflections of the other cases have not been presented due to the page limitations. When SSI is considered, lateral deflections and inter-storey drifts profoundly increase and performance level of the structure changes from life safe (less than 1.5% inter-storey drifts) to near collapse (less than 2.5% inter-storey drifts). Such a significance change in the inter-storey drifts and subsequently performance level of the model resting on soft soil deposit is absolutely dangerous and safety threatening. By taking local site effect into account,



(a)

Fig. 12 Inter-storey drifts of model S10 resting on soil class E_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake



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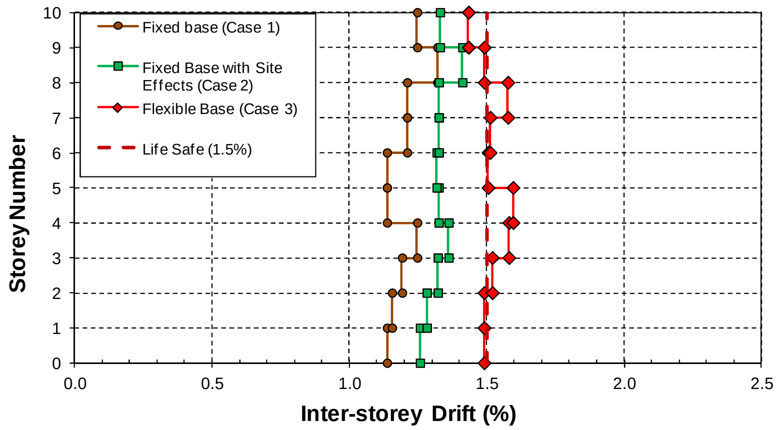
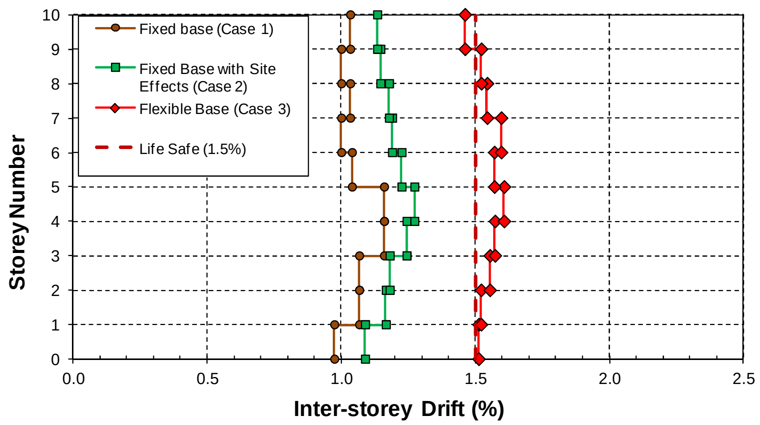
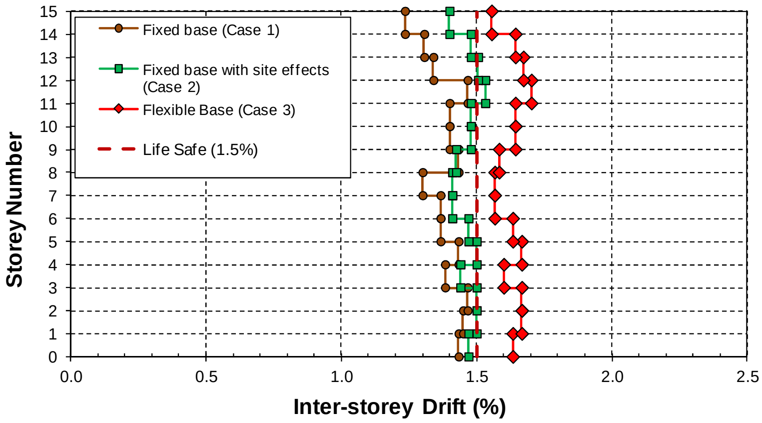


(c)

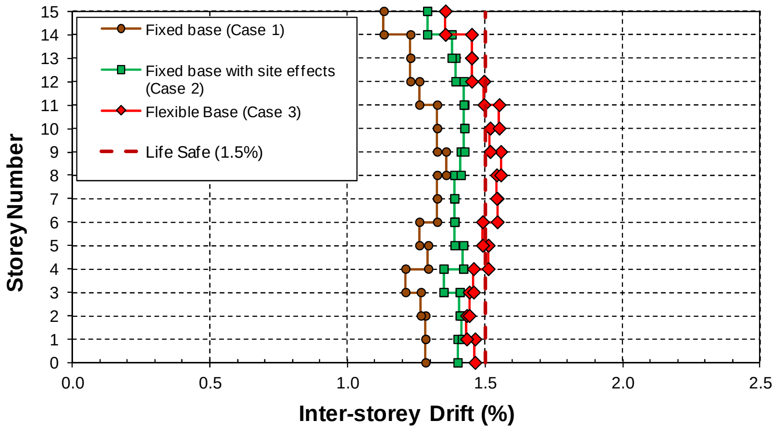


(d)

Fig. 12 Continued



(a)



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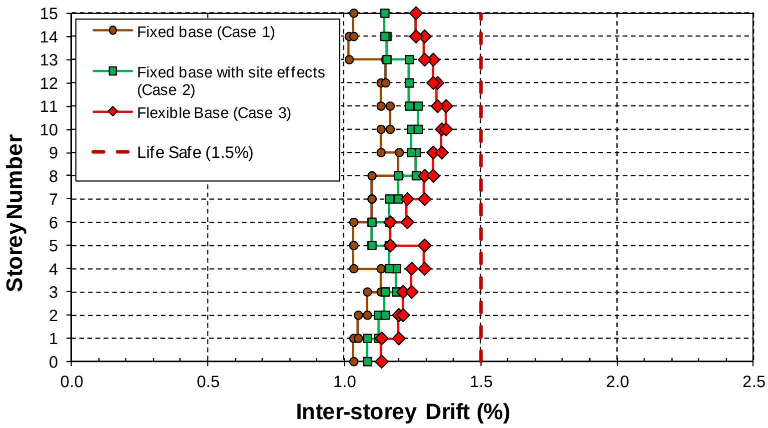
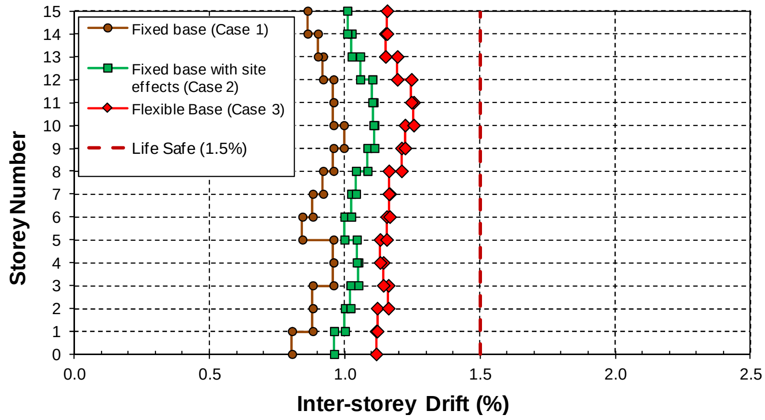


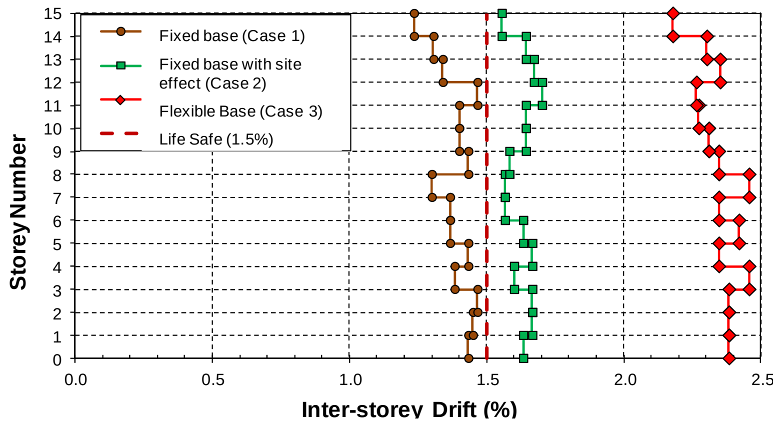
(c)

Fig. 13 Inter-storey drifts of model S15 resting on soil class D_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake

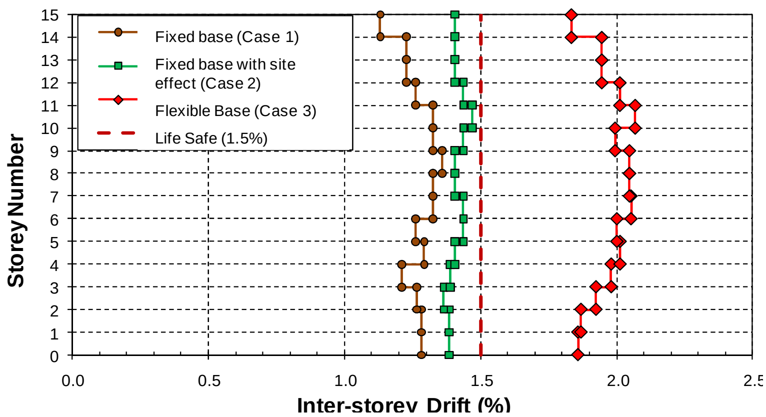


(d)

Fig. 13 Continued

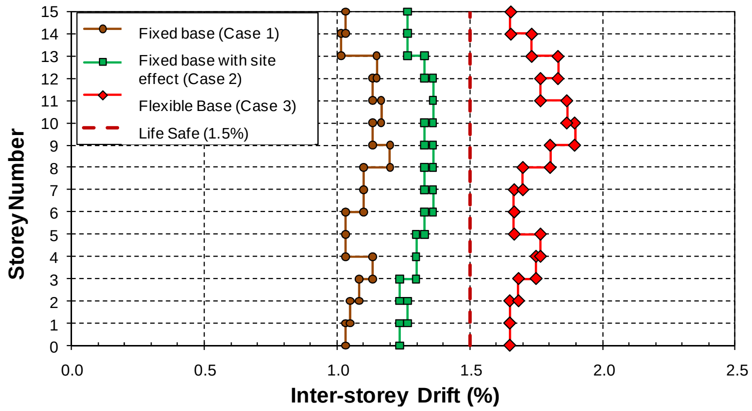


(a)

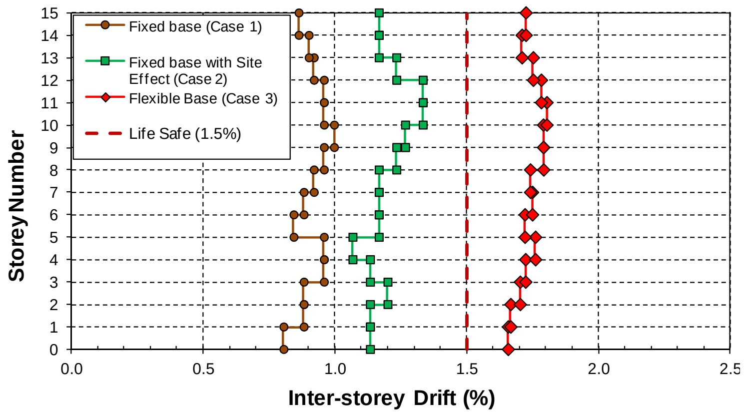


(b)

Fig. 14 Inter-storey drifts of model S15 resting on soil class E_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake

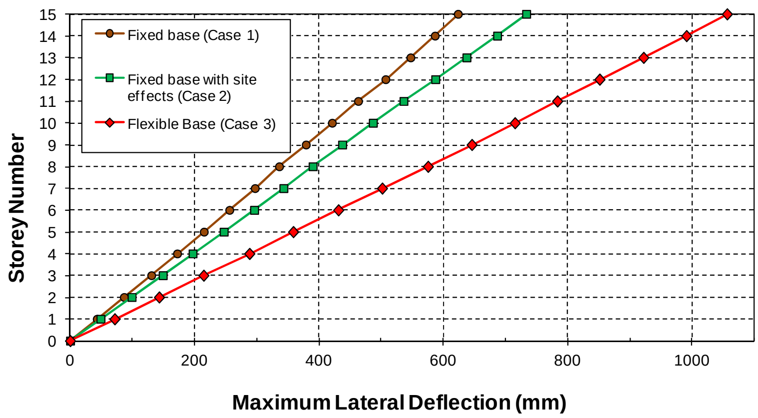


(c)



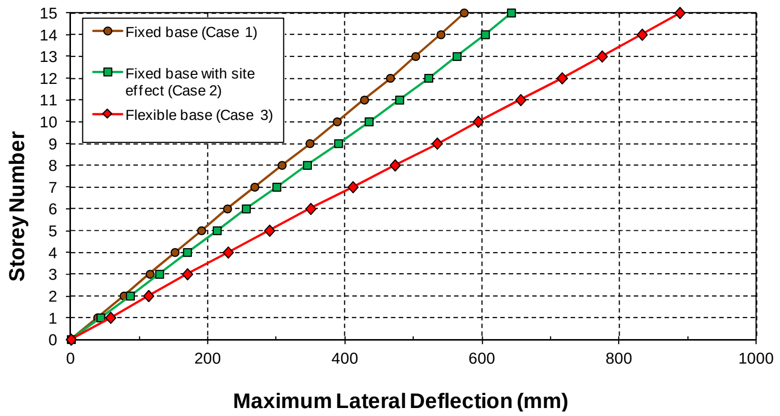
(d)

Fig. 14 Continued



(a)

Fig. 15 Lateral deflections of model S15 resting on soil class E_e for three different cases under the influence of; (a) Kobe (1995) Earthquake; (b) Northridge (1994) Earthquake; (c) El-Centro (1940) Earthquake; (d) Hachinohe (1940) Earthquake



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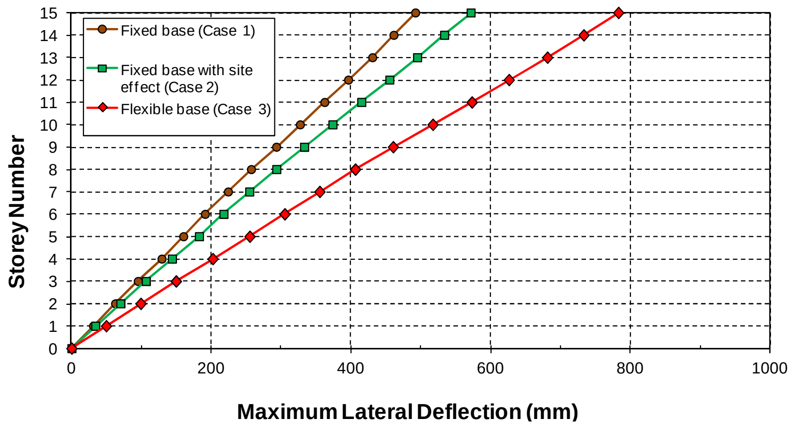

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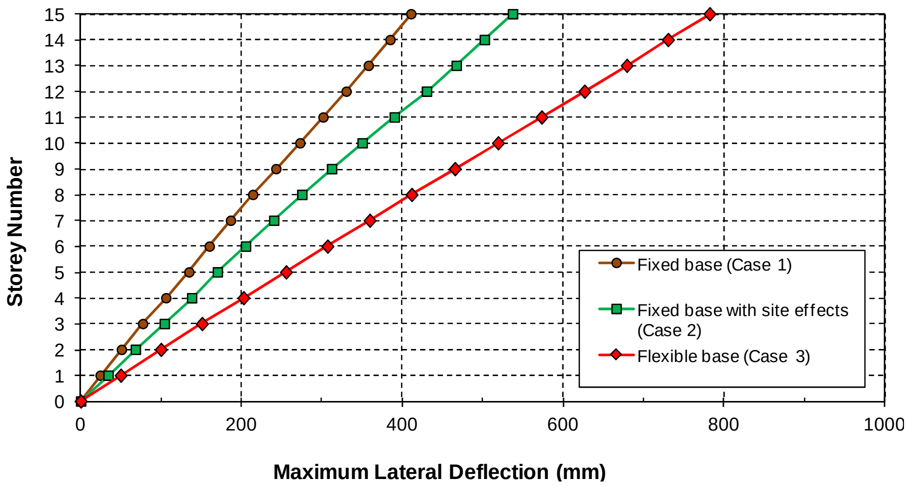

(d)

Fig. 15 Continued

ground motion intensity increases due to dynamic response of soft soil layer. Thus, base shear and inter-storey drifts increase accordingly. Considering SSI effects into account, the spectral acceleration and displacement change considerably with change in natural period, therefore, such increase in natural period noticeably alters the response of the building frames under the seismic excitation. In the case of mid-rise flexible structures, natural period relocates to the long period region of the response spectrum curve due to SSI. Hence, the acceleration response generally reduces while the displacement response tends to increase. By including soil-structure interaction effects into structural analysis, distortion portion of the structural lateral deformations, induced by the base shear, decreases while the total deformation increases. However, when only site effect is included, the base shear and corresponding lateral distortion incorrectly and unrealistically increase that never occurs in reality and jeopardises the above mentioned principle. Thus, for comparison purposes, site amplification excluding SSI cannot be used.

Rocking component plays an important role in lateral deformation of the superstructure. Relative lateral structural displacements under the influence of soil-structure interaction consist of rocking component and distortion component. Any change in the displacements is an outcome of changes in these components. Although Soil-Structure Interaction (SSI) reduces the base shear of the structure leading to the reduction in the structural distortion in comparison with fixed base structure, considering the effect of SSI increases the overall lateral deformation and consequently inter storey drifts of the structure mainly due to the rocking component. Moreover, in the seismic response of raft foundation, rocking and translation components are coupled and the response of the underneath soils to strong seismic shaking is strongly nonlinear.

7. Conclusions

Base on numerical investigations conducted in this study, the following conclusions can be drawn:

- It is understood that for the studied mid-rise structures resting on soil class D_e as well as 5 storey building frame resting on soil class E_e, by taking local site effect into account in the dynamic time history analyses, fairly realistic displacement response of the structures can be obtained without incorporating full dynamic soil-structure interaction in the analysis. However, the determined base shear results would be unacceptably over conservative and unrealistic predictions.
- For the studied mid-rise moment resisting frames higher that 5 storey (models S10 and S15) resting on relatively soft soil class E_e, it becomes apparent that local site effect contributes to the increase in the lateral deflection and inter-storey drifts. However, considering local site effect excluding soil-structure interaction does not warrant the safety of the structure. Inclusion of local site effect in the analysis of fixed base structures includes some increase in the lateral deflections and inter-storey drifts. However, this increase is only due to the increased base shear and not due to the changes of natural period and damping of the soil-structure system, which may be captured by consideration of SSI.
- The numerical results clearly indicate that the structural displacements and inter-storey drifts induced by SSI are larger than the corresponding values while only local site effect is included. As a result, analysis of moment resisting building frames higher than 5 storey resting on soil class E_e, considering the local site effect and excluding SSI may compromise safety of the structures.

- The site effect contributes to the increase in the base shear due to the earthquake amplification. However, soil-structure interaction causes a significant reduction in the base shear due to material and geometric damping as well as period lengthening of the whole system. Thus, the conventional inelastic design procedure by only including the local site effect (or using the earthquake record on the ground surface) excluding SSI is not adequate to guarantee the structural safety for the moment resisting buildings higher than 5 storey resting on soft soil deposits. It is highly recommended to practicing engineers working in high earthquake risk regions to consider full effects of soil-structure interaction rather than only applying site effect in dynamic time history analysis and design of fixed base mid-rise moment resisting building frames on soft soils.

References

ACI 318 (2008) *Building Code Requirements for Structural Concrete*, American Concrete Institute, Detroit, MI, USA.

Adam, M., Chouw, N. and Estorff, O. (2004), "Significance of soft local site in altering non-uniform ground motions and building response", *Al-Azhar Univ. Eng. J.*, **8**(1), 154-169.

AS1170.4 (2007), *Structural Design Actions – Part 4: Earthquake Actions in Australia*, Australian Standards, Sydney, Australia.

AS3600 (2009), *Concrete Structures*, Australian Standards, Sydney, Australia.

AS/NZS1170.1-2002 (2002), *Structural Design Actions – Part 1: Permanent, imposed and other actions*, Australian Standards, Sydney, Australia.

ATC-40 (1996), *Seismic Evaluation and Retrofit of Concrete Buildings*, Vol. 1, Applied Technology Council, Redwood City, CA, USA.

Borcherdt, R.D. (1994), "Estimates of site-dependent response spectra for design (Methodology and Justification)", *Earthquake Spectra*, **10**(4), 617-653.

BSSC (2003), *The 2003 NEHRP Recommended Provisions For New Buildings And Other Structures*, FEMA 450, Federal Emergency Management Agency, Washington D.C., USA.

Chandler, R., Clancy, J., Dixon, D., Goody, J. and Wooding, G. (2010), *Building Type Basics for Housing*, John Wiley & Sons, Inc., Hoboken, NJ, USA.

Fatahi, B. and Tabatabaiefar, S.H.R. (2013), "Fully nonlinear versus equivalent linear computation method for seismic analysis of mid-rise buildings on soft soils", *Int. J. Geomech.* 10.1061/(ASCE)GM.1943-5622.0000354

Fatahi, B., Fatahi, B., Le, T.M. and Khabbaz, H. (2013), "Small-strain properties of soft clay treated with fibre and cement", *Geosynthi. Int.*, **20**(4), 286-300.

FEMA 273 (1997), "NEHRP Guidelines for the Seismic Rehabilitation of Buildings", prepared by the Applied Technology Council for FEMA, Federal Emergency Management Agency , Washington, D.C., USA.

Furumura, T. and Kennett, B.L.N. (1998), "On the nature of regional seismic phases – III. The influence of crustal heterogeneity on the wave field for subduction earthquakes: The 1985 Michoacan and 1995 Copala, Guerrero, Mexico earthquakes", *Geophys. J. Int.*, **135**(3), 1060-1084.

Galal, K. and Naimi, M. (2008), "Effect of conditions on the response of reinforced concrete tall structures to near fault earthquakes", *Struct. Design Tall Special Build.*, **17**(5), 541-562.

Halbritter, A.L., Krutzik, N.J., Boyadjiev, Z. and Katona, T. (1998), "Dynamic analysis of VVER type nuclear power plants using different procedures for consideration of soil-structure interaction effects", *Nucl. Eng. Des.*, **182**(1), 73-92.

Hardin, B.O. and Drnevich, V.P. (1972), "Shear modulus and damping in soils: Design equations and curves", *J. Soil Mech. Found.*, **7**(8), 667-692.

Hokmabadi, A.S. Fatahi, B. and Samali, B. (2014), "Assessment of soil–pile–structure interaction influencing seismic response of mid-rise buildings sitting on floating pile foundations", *Comput. Geotech.*, **55**(1), 172-186.

IBC (2012), *International Building Code*, International Code Council (ICC).

Itasca Consulting Group Inc. (2008), *FLAC2D*: *Fast Lagrangian Analysis of Continua*, Version 6.0, User's manual, Minneapolis, MN, USA.

Karamodin, A.K. and Kazemi, H.H. (2008), "Semi-active control of structures using neuro-predictive algorithm for MR dampers", *Struct. Control Health Monitor.*, **17**(3), 237-253.

Kobayashi, H., Seo, K. and Midorikawa, S. (1986), "Estimated strong ground motions in the Mexico City earthquake", *The Mexico Earthquakes 1985*, *Factors Involved and Lessons Learned*, *Proceedings of the International Conference*, Camino Real Hotel, Mexico City, Mexico, pp. 97-111.

Kramer, S.L. (1996), *Geotechnical Earthquake Engineering*, Prentice Hall, Civil Engineering and Engineering Mechanics Series, ISBN 0-13-374943-6.

Matsui, J., Ohtomo, K. and Kanaya, K. (2004), "Development and validation of nonlinear dynamic analysis in seismic performance verification of underground RC structures", *J. Adv. Concrete Tech.*, **2**(1), 25-35.

Nateghi-A, F. and Rezaei-Tabrizi, A. (2011), "Nonlinear dynamic response of tall buildings considering structure–soil–structure effects", *Struct. Des. Tall Special Build.*, **22**(14), 1075-1082. DOI: 10.1002/tal.753.

Olsen, K.B., Pencman, J.C. and Schuster, G.T. (1995), "Simulation of 3-D elastic wave propagation in the Salt Lake Basin", *Bulletin of the Seismological Society of America*, vol. 85, pp. 1688-1710.

PEER (2012), *PEER Ground Motion Database*, Pacific Earthquake Engineering Research Centre, University of California, Berkeley, CA, USA.

Rahvar (2006a), *Geotechnical Investigations and Foundation Design Report of Kooh-e-Noor Commercial Building*, P.O. Rahvar Pty Ltd., Final Report, Tehran, Iran, pp. 1-69.

Rahvar (2006b), *Geotechnical Investigations and Foundation Design Report of Mahshahr Train Station*, P. O. Rahvar Pty Ltd., Iran Railways Authority, Mahshahr, Iran, pp. 1-42.

Rayhani, M.H. and El Naggar, M.H. (2008), "Numerical modelling of seismic response of rigid foundation on soft soil", *Int. J. Geomech.*, *ASCE*, **8**(6), 336-346.

Seed, H.B. and Idriss, I.M. (1969), "Influence of soil conditions on ground motion during earthquakes", *J. Soil Mech. Found. Div.*, *ASCE*, **95**(2), 99-137.

Sun, J.I., Golesorkhi, R. and Seed, B. (1998), *Dynamic Module and Damping Ratios for Cohesive Soils*, Earthquake Engineering Research Centre, Report No. UCB/EERC-88/15, University of California, Berkeley, CA, USA.

Tabatabaiefar, S.H.R., Fatahi, B. and Samali, B. (2013a), "Seismic behaviour of building frames considering dynamic soil-structure interaction", *Int. J. Geomech.*, **13**(4), 409-420.

Tabatabaiefar, S.H.R., Fatahi, B. and Samali, B. (2013b), "Lateral seismic response of building frames considering dynamic soil-structure interaction effects", *Struct. Eng. Mech.*, *Int. J.*, **45**(3), 311-321.

Veletsos, A.S. and Meek, J.W. (1974), "Dynamic behaviour of building-foundation system", *J. Earthq. Eng. Struct. Dyn.*, **3**(2), 121-138.

Vision 2000 (1995), "Performance Based Seismic Engineering of Buildings", Structural Engineers Association of California.

Wolf, J. (1985), *Dynamic Soil Structure Interaction*, Prentice Hall Co, NJ, USA.

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