

Three Dimensional Numerical Analysis of Seismic Soil-Structure Interaction Considering Soil Plasticity

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ABSTRACT

Effects of soil Plasticity Index (PI) on the seismic response of mid-rise buildings considering soil-structure interaction is investigated in this study. A 15-storey building resting on class E soil with different values of PI has been simulated utilising FLAC3D software. Fully nonlinear dynamic analysis under different earthquake recording including two near field and two far field earthquakes has been conducted and results in terms of the base shear, maximum lateral displacement and inter-storey drift are compared and discussed. Results indicate that as PI increases, the base shear of the structure resting on the soft soil, the maximum lateral displacement and inter-storey drift increase. It is concluded that practicing engineers should consider effects of soil plasticity on the seismic design of building frames constructed on soft soils accurately.

Introduction

During the recent decades, the importance of dynamic soil-structure interaction (SSI) has been well recognised in analysis of seismic response of superstructures. Several studies have shown that the superstructures are vulnerable to the effect of SSI, particularly when structures rest on soft soil deposits (e.g. Veletsos and Meek 1974; Hosseinzadeh and Nateghi 2004; Hokmabadi et al. 2014; Fatahi et al. 2014). When the soft soil deposit is subjected to strong seismic loading, significant soil damping is induced by soil modulus degradation and inertial interaction becomes predominant, causing excessive displacements near the ground surface. This behaviour of soft soil, which can be described in the forms of backbone curves, is nonlinear and hysteretic. The curves expressing shear modulus and damping ratio as a function of shear strain vary with soil characteristics, which significantly influence the structural response considering SSI. Moreover, Vucetic and Dobry (1991) concluded that plasticity of soils plays an important role in determining backbone curves in cohesive soils. Plasticity is an important characteristic of fine soils, which indicates the ability of a soil to deform irreversibly without cracking or crumbling. The plasticity of a soil can be described by Plasticity Index (PI) which indicates the range of water content where a soil exhibits plastic behaviour under stress. The lower and upper limits of the range of water content over which the soil behaves plastically are defined as Plastic Limit (PL) and Liquid Limit (LL), respectively, while the range of water content can be defined by Plasticity Index (PI) as follows:

$$PI = LL - PL \quad (1)$$

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Dynamic Behaviour of Cohesive Soils

Figure 1 shows the nonlinear stress-strain relationship of typical soils subjected to the cyclic loading. Two important characteristics of hysteresis loop, which are inclination and breath are used to describe the hysteresis response of soils. According to Kramer (1996), the inclination of the loop represents stiffness of the soil, which can be described at any point during the loading process by the tangent shear modulus G_{tan} . It is clear that the tangent shear modulus varies throughout a cycle of loading, but its average value over the entire loop can be approximately represented by the slope of the line connecting the origin to the tip of the loop and the slope defines the secant shear modulus G_{sec} .

$$G_{sec} = \frac{\tau_c}{\gamma_c} \quad (2)$$

where, τ_c and γ_c are the shear stress and shear strain amplitudes at the defined point, respectively. Therefore, G_{sec} represents the inclination of the hysteresis loop. The breath of the hysteresis loop, which is related to the area within the loop, represents the absorbed energy, and damping ratio ξ can be defined as below:

$$\xi = \frac{W_D}{4\pi W_S} = \frac{1}{2\pi} \frac{A_{loop}}{G\gamma_c^2} \quad (3)$$

where, W_D is the absorbed energy in one loop, W_S is the maximum strain energy created by the loop, and A_{loop} is the area of the hysteresis loop which is equal to W_D . The parameters G_{sec} and ξ are used to describe the cyclic behaviour of the soil in the equivalent linear analysis and often referred to as the soil equivalent linear parameters.

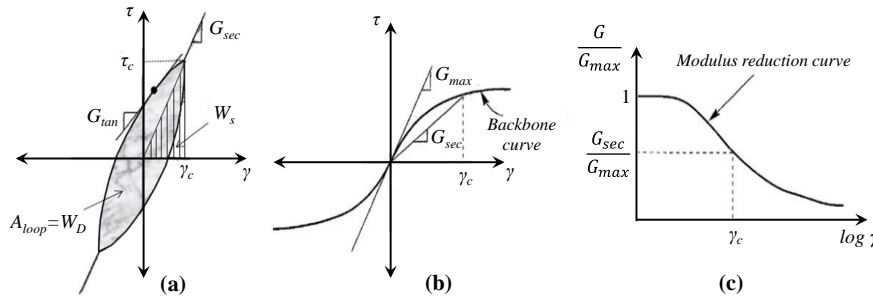


Figure 1 (a) Hysteretic stress-strain relationship; (b) Backbone curve; and (c) Typical modulus reduction curve for soils (After Kramer, 1996)

The secant shear modulus of the soil element changes with the cyclic shear strain amplitude, which is large at low strain amplitudes and decreases as the shear strain amplitude increases. As shown in Figure 1b, the locus of tips of the hysteresis loops of different cyclic shear strain amplitudes form the backbone curve which matches the monotonic loading curve for the same type of soil and the slope of the backbone curve at the origin represents the maximum value of the shear modulus, G_{max} . At greater shear strain amplitudes, the modulus ratio, G_{sec}/G_{max} drops to values less than one. Therefore, to precisely represent the behaviour of the soil under the cyclic loading, both G_{max} and modulus reduction curve are required.

Vucetic and Dobry (1991) conducted a review on the available cyclic loading test results and concluded that the Plasticity Index (PI) is the main factor controlling the variations of the shear modulus reduction and damping ratio against the cyclic strain curve for a wide variety of cohesive soils. Solid lines in Figure 2a and 2b illustrate the ready-to-use charts provided by Vucetic and Dobry (1991) for modulus degradation and damping ratio, respectively. The aim of these two charts is to provide a design tool for practicing engineers since PI is readily available. As observed in Figure 2, when PI increases, G_{sec}/G_{max} increases while damping ratio decreases.

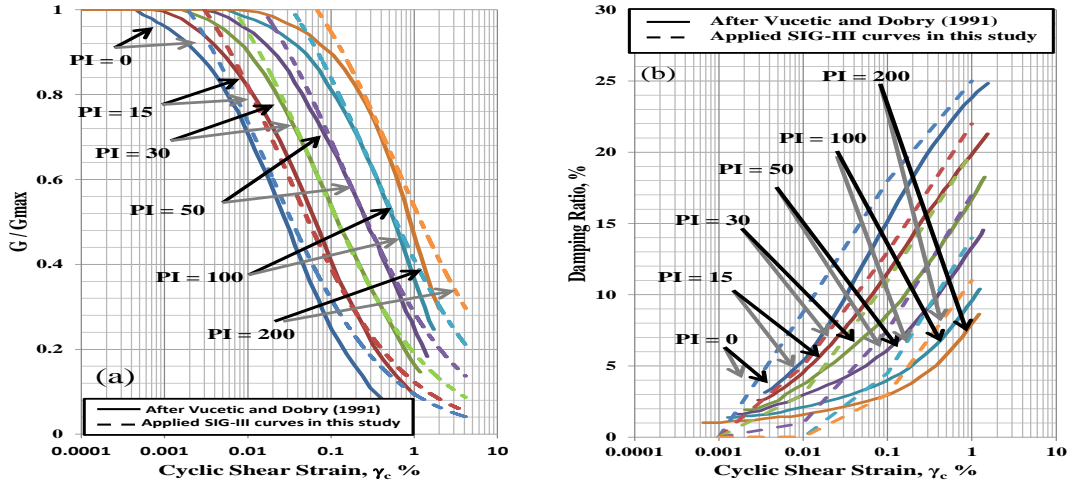


Figure 2 (a) Relation between G/G_{max} versus cyclic shear strain for cohesive soils; (b) Relations between damping ratio versus cyclic shear strain for cohesive soils

Numerical Modeling

In this study, dynamic seismic soil-structure interaction analysis is conducted adopting direct method which evaluates the dynamic response in a single step, as it can perform fully nonlinear analysis. Furthermore, time domain analysis as recommended by Chu (2006) necessary to compute the nonlinear dynamic response is utilised. In order to have realistic fully nonlinear analysis, a three dimensional explicit finite difference based program FLAC3D (Fast Lagrangian Analysis of Continua) version 5.0 has been employed. In this program, behavior of different types of materials according to their prescribed constitutive models in response of applied loads and boundary conditions can be simulated. For a dynamic analysis, damping in the numerical simulation should be reproduced in magnitude and pattern related to the energy losses in the system when subjected to a dynamic loading. In soil and rock, natural damping is mainly hysteretic. Hysteretic damping algorithm is incorporated in FLAC3D dynamic analysis to simulate the realistic behavior of soils. Modulus degradation curves imply nonlinear stress-strain curves. In case of an ideal soil in which the stress depends only on the strain, an incremental constitutive relation from the degradation curve can be described by the strain-dependent normalised secant modulus (M_s) as follows:

$$M_s = \frac{\bar{\tau}}{\gamma} \quad (4)$$

where, $\bar{\tau}$ is the normalised shear stress which can be obtained through local shear stress divided by the initial shear modulus and γ is the shear strain. The normalised tangent modulus (M_t) is then obtained as follows:

$$M_t = \frac{d\bar{\tau}}{d\gamma} = M_s + \gamma \frac{dM_s}{d\gamma} \quad (5)$$

The incremental shear modulus in a nonlinear simulation is then given by G_{sec} , where G_{sec} is the given shear modulus obtained from Equation (2). The formulations described in Equations (4) and (5) are implemented in FLAC3D, by modifying the strain rate calculation so that the mean strain rate tensor (averaged over all subzones) is calculated before any calls are made to the constitutive model functions. At this stage, the hysteretic logic is invoked, returning a modulus multiplier, which is passed to any called constitutive model. The model then uses the multiplier M_t to adjust the apparent value of the tangent shear modulus of the full zone being processed. In this study, the tangent modulus model named SIG-III implemented in FLAC3D is employed to simulate hysteretic behavior in the soil deposit. The

mathematical formulation of the model is defined as:

$$M_s = \frac{a}{1 + \exp\left(-\frac{\log_{10}(\gamma) - x_0}{b}\right)} \quad (6)$$

where, M_s is the secant modulus (G/G_{max}), γ is the cyclic shear strain, and a , b and x_0 are the model parameters. By adopting different model parameters, this model is able to generate different backbone curve for different types of material in dynamic analysis.

Earthquake ground motions

In order to perform a comprehensive study on the seismic response of the structural models, two near field seismic accelerations which are Northridge, 1994 and Kobe, 1995 and two far field seismic accelerations which are El-Centro, 1940 and Hachinohe, 1968 shown in Figure 3 are utilised in the time history analysis, which are selected by the International Association for Structural control and Monitoring for benchmark seismic studies (Karamodin and Kazemi 2008) and Figure 4 illustrates the spectral accelerations for each of earthquake input.

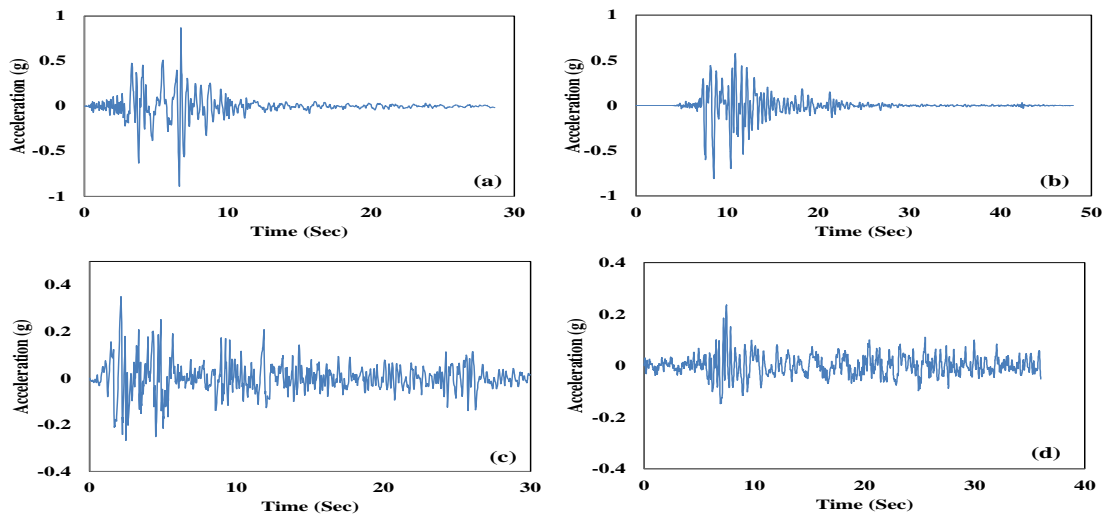


Figure 3 (a) Northridge Earthquake 1994; (b) Kobe Earthquake 1995; (c) El Centro Earthquake 1940; (d) Hachinohe Earthquake 1968

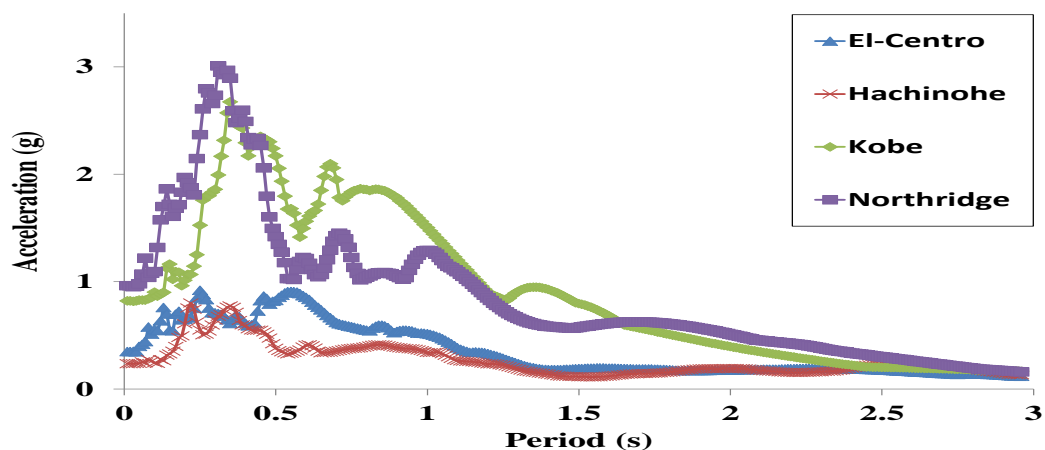


Figure 4 Spectral accelerations of four input earthquake with 5% damping ratio

Structural models and soil deposit

Figure 5 shows the dimensions of the structure adopted in this study, whose natural period is

1.28 seconds. SAP2000 V14 has been utilised for the structural design purpose. All the structural sections of the model have been designed based on the inelastic method assuming elastic - perfectly-plastic behavior. According to AS/NZS1170.1-2002 (Permanent, imposed and other actions), permanent and imposed loads are determined and applied to the structure model. It should be noted that cracked sections for the reinforced concrete sections are taken into account by multiplying second moment of area of the uncracked sections (I_g) by cracked section coefficients ($0.35I_g$ for beams, $0.70I_g$ for columns and $0.25I_g$ for slabs) according to ACI318-08 (2008). The model foundation is a square shallow reinforced concrete foundation which is 14 meters in length and width, and 1 meter in depth. The entire numerical model has been illustrated in Figure 5. Then four earthquakes have been applied at the base. Finally, the 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 drift to 1.5%.

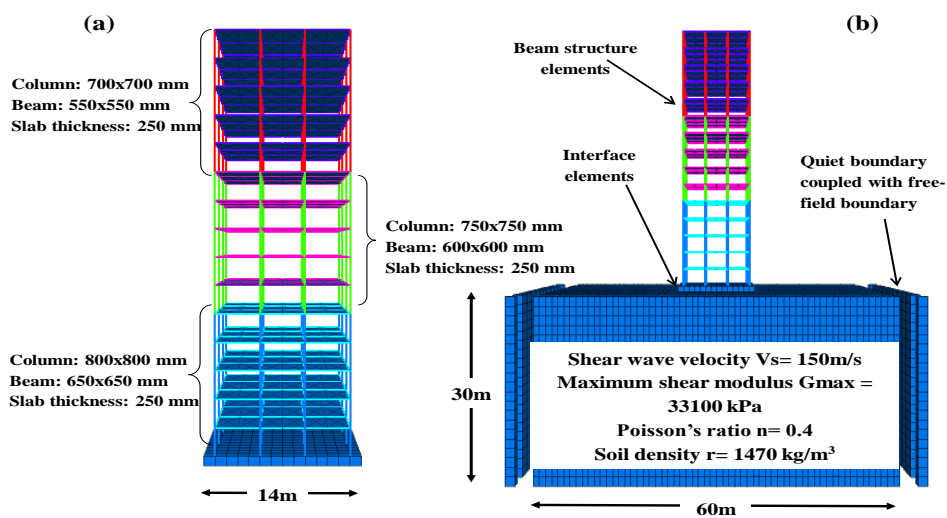


Figure 5 (a) Fixed base model; (b) Flexible base model utilised in FLAC 3D simulation

According to the classification of AS1170.4-2007 (Earthquake actions in Australia), a soft soil representing subsoil class E has been selected in this study. Figure 3b shows the characteristics of the soil and dimensions adopted in this study. Different sets of SIG-III model parameters a , b and x_0 have been determined in order to regenerate backbone curves reported by Vucetic and Dobry (1991) to be used in the numerical simulation. Table 1 summarises the determined values of the model parameters.

Table 1 SIG-III model parameter for various Plasticity Indices

Plasticity Index(PI)		PI=0	PI=15	PI=30	PI=50	PI=100	PI=200
SIG-III model parameter	a	1.25	1.25	1.25	1.25	1.25	1.25
	b	-0.7	-0.7	-0.7	-0.7	-0.7	-0.7
	x_0	-1.75	-1.55	-1.2	-0.85	-0.5	-0.2

The interface elements are applied between the superstructure and the soil deposit to simulate the real behaviour between the structure and the subsoil. 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. Obviously, the relative interface movement is controlled by interface stiffness values in the normal and tangential directions. Normal and shear spring stiffness values for interface elements of the soil-structure model are set to ten times the equivalent stiffness of the neighbouring zone based on the

recommendation of Itasca FLAC3D manual (2014).

$$k_s = k_n = 10 \left[\frac{(K + \frac{4}{3}G)}{\Delta z_{\min}} \right] \quad (7)$$

where, K and G are bulk and shear modulus of the neighbouring zone, respectively, and Δz_{\min} is the smallest width of an adjoining zone in the normal direction. Additionally, the interface elements have been applied in the way that the foundation can separate from the soil and gapping is allowed during the analysis. For side boundaries of the soil deposit, free field boundaries have been employed. Free-field boundaries have been simulated using a developed technique, involving one-dimensional free-field wave propagation in parallel with the main-grid analysis (Itasca, 2014). Thus, plane waves propagating upward undergo no distortion at the boundaries because the free-field grid supplies conditions identical to those in an infinite model. In addition, a rigid boundary as the bottom of the soil deposit has been adopted in this study.

Results and discussions

The results of dynamic analyses for 15 storey structural models in terms of the maximum base shear, lateral deflection and inter-storey drift under the influence of four earthquake records for fixed base condition and cases considering soil-structure-interaction with different Plasticity Indices are derived from FLAC3D history records and compared in Tables 2 - 3 and Figure 6.

According to Table 2, it is observed that the base shear of the structures modeled considering SSI is always less than the base shear of the corresponding fixed base cases. Comparing the base shear results, it is evident that as the PI of the subsoil increases from 0 to 200, the base shear increases. As observed in Figure 2, by increasing the soil plasticity, the stiffness of the subsoil increases, while damping ratio decreases. Thus less distortion of the soil occurs as PI increases and consequently more energy transfers through the soil into the system. Thus, the increase of PI evidently leads to increase in the base shear of the building resting on the soft soil deposit.

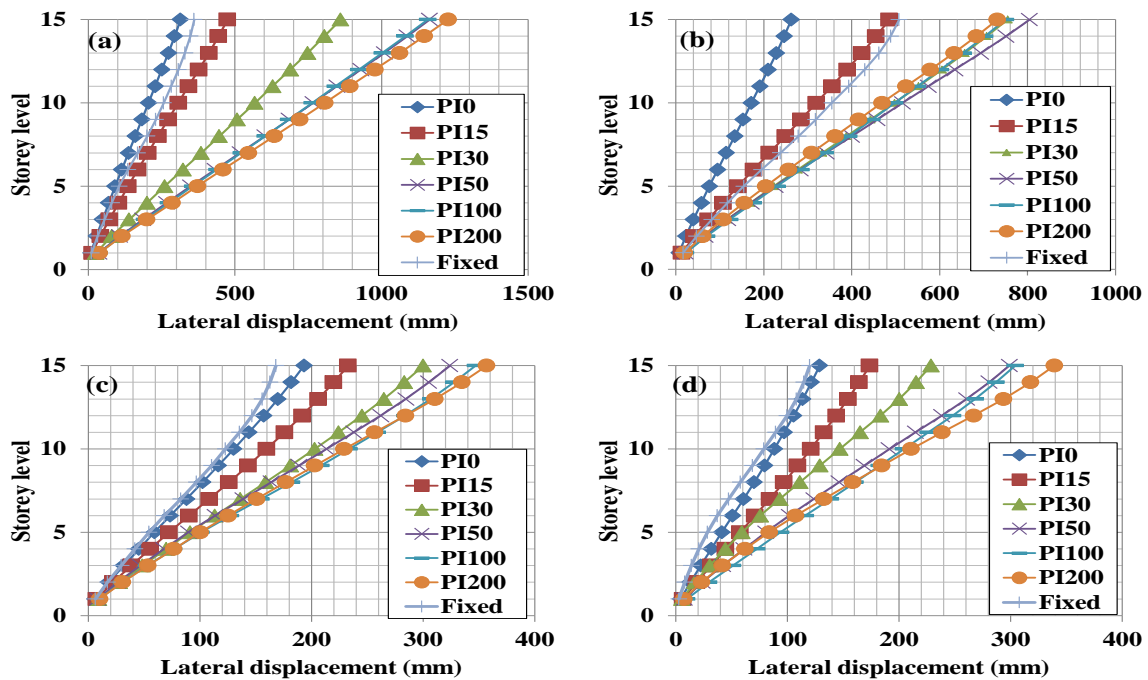


Figure 6 Maximum lateral displacement (a) Northridge1994; (b) Kobe1995; (c) El-Centro1940; (d) Hachinohe1968

Table 2 Maximum base shear results obtained from different cases

Earthquake	Base shear kN						
	PI=0	PI=15	PI=30	PI=50	PI=100	PI=200	Fixed base
Northridge	7068	12611	14952	25067	26855	27068	36653
Kobe	6966	9977	11765	14666	19607	23314	36127
El-Centro	5148	8470	9200	11089	12062	12103	14345
Hachinohe	4382	7267	10668	10640	11723	12474	13727

Table 3 Maximum Inter-storey drift reported for different cases

Earthquake	Maximum inter-storey drift (%)						
	PI=0	PI=15	PI=30	PI=50	PI=100	PI=200	Fixed base
Northridge	0.77	1.18	2.07	2.84	2.76	2.92	1.10
Kobe	0.65	1.20	1.84	2.00	1.83	1.84	1.42
El-Centro	0.48	0.60	0.76	0.84	0.91	0.92	0.48
Hachinohe	0.33	0.44	0.61	0.78	0.75	0.94	0.36

Comparing the results in Figure 6 and Table 3 both taken when the maximum lateral displacement occurred at top floor, it becomes evident that when $PI = 0$ (e.g. low plasticity silt), the lowest maximum lateral displacement is observed (even lower than the corresponding value for the fixed base case), while for $PI = 200$ (e.g. high plasticity clay), the highest maximum lateral displacement is achieved. The displacement increases as PI increases (except Kobe case, where due to the change in the system period, the second mode could be activated with extreme strong base shear at a certain input frequency). Although the shear wave velocity (or G_{max}) is constant, the performance of the soil deposits is significantly influenced by PI , due to the combined effects of shear modulus degradation and damping ratio. Moreover, as the damping reduces due to the increase in the values of PI , the magnitude of acceleration reaching the structure and dominant frequency increase, which is less than dominant frequency of input acceleration for all cases. Thus, it is reasoned to conclude that due to the decrease in damping, the response of structure increases, which eventually indicates the variation of the damping ratio induced by PI has a dominant effect on the system behaviour. In addition, considering the conservation of energy, in relatively stiff soil deposits experiencing less soil distortion and damping during earthquake, more energy may transfer into the structure resulting in increased foundation rocking and structural displacement. Generally, for soils with low Plasticity Index (e.g. low plasticity silt), the maximum lateral displacement and inter-storey drift of the structure built on the soft soil are less than the corresponding values for the fixed base structure. In contrast, when soil has high plasticity (e.g. high plasticity clay), the maximum displacements and inter-storey drifts have been amplified significantly, especially, for the near field earthquakes (Northridge and Kobe). In fact, for both near field earthquakes, the maximum recorded inter-storey drifts considering SSI are well more than 1.5% (life save level), which is extremely dangerous and safety threatening.

Conclusions

In this study, the influence of Plasticity Index (PI) variations ($PI=0$ 200) on the seismic response of mid-rise buildings has been numerically investigated. A 3D numerical soil-structure model has been developed and employed utilising FLAC3D adopting direct method of analysis. In order to capture soil nonlinearities, SIG-III model (Eqn. 6) has been used to simulate backbone curves of the shear modulus and the damping ratios versus the shear strain for soils with different Plasticity Indices. Numerical results show that as the Plasticity Index of subsoil increases (G/G_{max} increases and damping ratio decreases), the base shear, the maximum lateral displacement and the maximum inter-story drift increase. The amplification

of the lateral displacement and internal drifts could potentially change the performance level from life safe to total collapse which is safety threatening. It can be concluded that soil-structure interaction has considerable effects on the seismic response of mid-rise building frames resting on soft soil deposits and increase of the Plasticity Index could considerably amplify response of the structure. Thus, conventional design procedure excluding soil-structure interaction and soil plasticity may not be adequate for the safe design of mid-rise buildings resting on soft soils. Furthermore, in order to obtain reliable results, the influence of Plasticity Index should be taken into account while conducting soil-structure interaction analysis.

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