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MODELLING OF SOIL DAMPING FOR SEISMIC GROUND RESPONSE BY NONLINEAR FINITE ELEMENT ANALYSIS

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Abstract

In this paper, the nonlinear dynamic behavior of clayey soil using 2D finite element analyses is presented. This paper presents comparative analyses of seismic response by different numerical approaches. The behavior of two types of soil represented the soft and stiff clay is investigated under effect of three different acceleration time histories. The amplification functions of seismic signals obtained by 1D equivalent linear visco-elastic analyses performed in the frequency domain is used to evaluate the initial values of Rayleigh damping (αR and βR) coefficients to perform the nonlinear finite element analyses. The influences of type of soil and peak ground acceleration on the nonlinear dynamic behavior of soil are studied. The results show a contraction of peak ground acceleration profile and the spectra as compared to the equivalent linear analysis especially in the uppermost portion of the deposit. The rate of increase of shear strain during ground shaking and the permanent shear strain increases as increase of peak ground acceleration for soft soil and linearly for stiff clay. The results obtained by the results indicate that the equivalent linear analysis should not be considered as a right way to modeling strong motion earthquakes especially for soft clay deposit.

Keywords: Seismic ground response analysis, Rayleigh damping, finite element analysis, nonlinear analysis

1. Introduction

There are two main numerical methods to solve the wave propagation problem namely linear or equivalent linear analysis method (frequency domain solution) and nonlinear analysis method (time domain solution). However the equivalent linear analysis is widely used in engineering practice due its simplicity [1-2]. It is essentially a linear method does not account for the change in soil properties during the ground motion. In the current study the nonlinear analysis is used to investigate the behavior of soil throughout the earthquake duration uses a step by step integration scheme and more accurately simulates the true nonlinear behavior of soils.

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Nowadays, the finite element analysis is available to solve the wave propagation problems. The behavior of soil can be analyzed using linear, equivalent linear or nonlinear constitutive models. Permanent strain in soil subjected to earthquake forces was also obtained using non linear finite element analysis. The stresses, deformations and the force acting on the structural elements that interacts with the soil can be predicted in one single analysis [3-6]. Various approaches used based on finite element method mainly vary with the constitutive model adopted to model the behavior of soil [7-10]. Park and Hashash [11] used nonlinear time-domain site response analysis to capture the soil hysteretic response and nonlinearity due to medium and large ground motions. Soil damping is captured through the hysteretic energy dissipating response, and one can use their proposed formulations in nonlinear site response analysis. Ciro Visone et al. [12] presented a comparative study on frequency and time domain analyses for the evaluation of the seismic response of subsoil to the earthquake shaking by different computer programs. Amorosi et al. [13] performed 2D finite element analysis of seismic ground response of a clayey deposit, using linear visco-elastic and visco-elsto-plastic constitutive models, the viscous and linear elastic parameters are selected according to a novel calibration strategy, leading to FE results comparable to those obtained by 1D equivalent-linear visco-elastic frequency-domain analyses.

It is well known that the linear and equivalent linear analysis provides the reasonable results for seismic ground problems but these methods remain approximate methods to simulate the actual nonlinear processes [14]

The current study presents an appropriate method to model the soil damping parameters in 2D nonlinear analysis performed in time domain. Also, this paper presents A comparative study on a set of 1D ground response analyses performed in the frequency domain using commercial computer program DEEPSOIL [15] with the corresponding nonlinear analyses based on 2D finite element obtained by PLAXIS program [16]. The nonlinear analysis is used to investigate the relationship between maximum shear strain during earthquake motion and permanent shear strain with the peak ground acceleration.

The analyses are performed on two soil profiles under the effect of three input seismic motions. the amplification functions of the signals obtained by 1D equivalent linear visco-elastic analyses is used to evaluate the initial profiles of Rayleigh damping coefficients in the nonlinear finite element analysis as shown later. Also the nonlinear analysis is used to investigate the relationship between maximum shear strain during earthquake motion and permanent shear strain with the peak ground acceleration.

2. Soil profile

Two ideal 60m thick soil deposits were considered in the current analyses. The first deposit consists of single stratum of soft clay, while the second deposit is composed of stiff clay. The physical and mechanical parameters for both strata are shown in table 1. The shear wave velocity of soils varies in proportion to (pm)1/4 where pm is effective mean confinement pressure. The profile of small strain shear stiffness G_0 with depth was calculated by following equation:

$$G_0 = \rho\, Vs^2 \qquad (1)$$

where the parameter ρ is the mass density of soil. The variation of G_0 with depth and corresponding shear wave velocity V_s are reported in Fig. 1. In both deposits, the water table was assumed at the ground level and the small strain damping was considered constant with depth.The variation of G/G_0 and D/D_0 with shear strain level γ was defined according the results reported in the literature [17] as function of Ip as shown in Fig. 2.

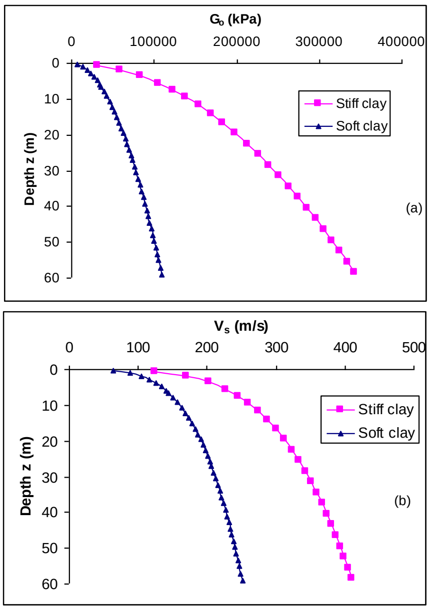

Fig. 1. Soil profile. (a) Profile of the small strain shear modulus ; b) Profile of the shear wave velocity

Table 1.
Physical and mechanical parameters of assumed soil deposit.

Parameters	Soil profile	
	Stiff clay	Soft clay
Plasticity index I_p (%)	42	45
Unit weight of volume γ (kN/m^3)	20	17
Coefficient at rest earth pressure k_0	0.61	0.66
Friction angle ϕ'(°)	23	20
Poisson's ratio ν'	0.25	0.25
Cohesion c' (kPa)	40	10
Small-strain damping ratio D_o (%)	1	1
Reference Secant modulus E_{50}^{ref} (kPa)	60000	35000
Reference oedometer modulus E_{oed}^{ref} (kPa)	60000	35000
Reference unloading reloading modulus E_{ur}^{ref} (kPa)	180000	109000
Reference confining pressure p^{ref} (kPa)	100	100

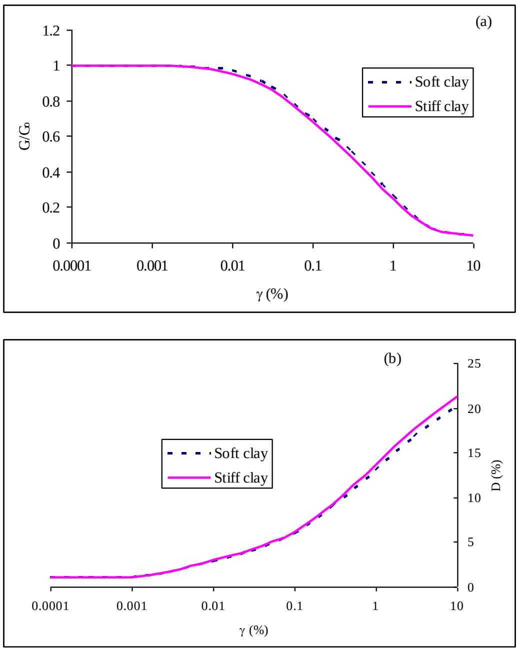

Fig. 2. a) Modulus reduction curve G/G_0; b) variation of damping ratio D with shear strain γ

3. Seismic input motions

In numerical computation, the earthquake loading is often imposed as an acceleration time-history at the base of the model. To investigate the influence of input motion on the nonlinear seismic response of soil layer, three different acceleration time histories were considered, Fig. 3. These motions are obtained using the data base available in DEEPSOIL Hashash et al. 2008 [15] which were effectively used earlier by Choudhury and Savoikar 2009 [2] for equivalent-linear ground response analysis of municipal solid waste material. The earthquake characteristics of these motions like peak ground acceleration, Predominant period and significant duration are also presented in Table 2. These are derived using Seismo Signal program (see www.SeismoSoft.com).

In equivalent linear analysis, the elastic bed rock was assumed. The main characteristics of elastic bed rock are illustrated in table 3.

The input seismic signals were considered as applied at the rock outcropping of the deposit. Indeed these earthquake signals are measured at the ground surface but for simplicity we dealt with these motions as artificial earthquake at the bed rock. The corresponding bed rock motions were then calculated by performing an equivalent-linear analysis. The corresponding bed rock motions for two soil profile performed in 2D analysis are shown in Fig. 4 and Fig. 5.

Table 2.
Main characteristics of the input motions

Earthquake	PGA (g)	Predominant period (s)	Significant duration (s)
Motion 1	0.442	0.38	3.70
Motion 2	0.278	0.3	11.57
Motion 3	0.119	0.118	6.19

Table 3.
Elastic bed rock parameters used in the analyses

Parameter	Value
Mass density (kg/m^3)	2038
Unit weight (kN/m^3)	20
Shear wave velocity V_s (m/s)	1200

4. Numerical models

Two types of analyses were performed in the current study using two numerical codes. The first type is one dimensional analysis adopting frequency domain analysis using the equivalent linear visco-elastic code DEEPSOIL [15].

The DEEPSOIL code is widely used for ground response analysis or soil amplification studies as it provides reasonable estimates of ground motion [2]. It is a program for one dimensional site response analysis that performs frequency domain for linear and equivalent linear analysis and time domain for nonlinear analysis. The DEEPSOIL code was used here to predict the ground response adopting the equivalent linear analysis. The equivalent-linear model assumes that the shear modulus G and damping ratio D are function of the shear strain amplitude γ. The equivalent linear analysis was based on the pioneering work of Idriss and Seed [18], and Seed and Idriss [19] as employed in the widely used program SHAKE [20]. In DEEPSOIL analyses, the profile of small strain stiffness shown in Fig. 1 were discretised by constant stiffness sub-strata of thickness ranging from 3m at the base of stratum to 1m at the surface. The equivalent linear model employs an iterative procedure in the selection of the shear modulus and damping ratio.

Two dimensional finite element model is performed in the second type of analysis using the PLAXIS code V.8.2 [16]. This code is a commercial finite element program that allows performing stress strain analysis for various types of geotechnical problems. The earthquake analysis can be performed by imposing an acceleration time history at the base of the two dimensional finite element model and solving the equations of motion in time domain by adopting a Newmark type implicit time integration scheme.

In nonlinear analysis the soil was modelled by 15 node triangular finite element. The hardening soil model (hyperbolic stress–strain relation) was used in order to simulate the nonlinear behavior of soil. Hardening-Soil model is the hyperbolic relationship between the vertical strain, ε_1, and the deviatoric stress, q. The analyses were performed under undrained conditions. In Plaxis program it is possible to specify undrained behavior in an effective stress analysis using effective model parameters [16] [5], [13]. The choice of boundary conditions influences the amount of energy dissipation due to the wave propagation in the ground. The position of the boundary and the kind of mechanical constraints should reproduce, at best, the energy transmission outwards the computation domain. Viscous adsorbent boundaries based on the method described by Lysmer and Kuhlemeyer [21] are a rather widespread procedure. In this case, normal and tangential stress components adsorbed at the boundary location are:

$$\sigma_n = -C_1 \rho V_p \dot{u}_n \qquad (2)$$
$$\tau = -C_2 \rho V_s \dot{u}_t \qquad (3)$$

where ρ is the density of the material, V_p and V_s are the compression and shear wave velocities, $\dot{u}_n$ and $\dot{u}_t$ are the normal and tangential components of the velocity, C_1 and C_2 are relaxation coefficients. Some suggestions exist in literature for the choice of these parameters. The parameters C_1 and C_2 are assumed here 1 and 0.25 respectively. The bottom of the mesh is assumed to be rigid. The model of dynamic analysis can be sketched in Fig. 6. The characteristic dimension of the element h always satisfies the condition $h \le h_{max} = V_s/(6 \div 7) f_{max}$

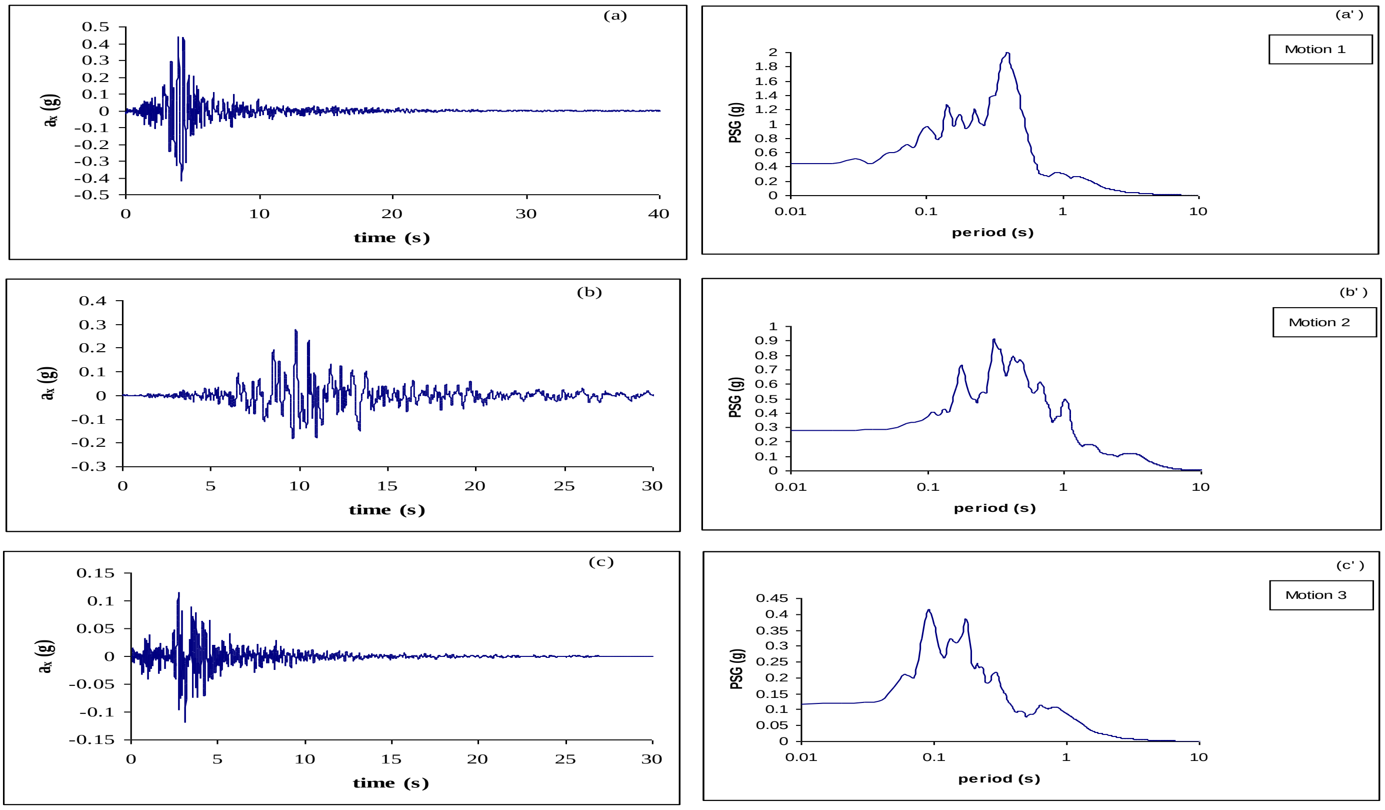

Fig. 3. Seismic input signal of the three selected acceleration time histories and corresponding response spectrum: (a) Motion 1, (b) Motion 2, (c) Motion 3.

where V_s is the shear wave velocity and f_{max} is the maximum frequency of the seismic signal.

The two lateral domains, characterized by a coarse mesh, to reduce the computational costs [5,13]. It is characterised by width equal to eight times its height, in order to minimize the effect of boundary conditions on the computed results [23]. The generalized Newmark method [22] is adopted for the time integration under dynamic conditions. The following values of the Newmark parameters were selected in all the analyses illustrated in this paper: $\alpha_N = 0.3025$ and $\beta_N = 0.6$.

In the PLAXIS code, the Rayleigh damping formulation is implemented and the values of α_R and β_R are obtained by:

$$\begin{Bmatrix} \alpha_R \\ \beta_R \end{Bmatrix} = \frac{2D}{\omega_m + \omega_n} \begin{Bmatrix} \omega_n \omega_m \\ 1 \end{Bmatrix} \quad (4)$$

where ω_m and ω_n are the angular frequencies related to the limits of frequency interval (f_m,f_n) over which the viscous damping is equal to or lower than D.

This paper supposes that, the values of α_R and β_R for nonlinear analysis are chosen according to Eq.4 for the frequency interval (f_m,f_n) depending on the damping coefficient predicted from the equivalent linear analyses

5. Calibration of damping ratio for nonlinear analysis

In fact it is well known that the damping ratio depends on the level of shear strain. In time domain schemes there are two sources of damping: viscous damping, generally introduced through the Rayleigh [24] formulation, and the hysteretic dissipation associated to the irreversible material response. In order to simulate the wave propagation problem through the nonlinear finite element analysis, this paper supposes that the viscous damping ratio that implemented in nonlinear analysis depends on the values of Reyleigh damping coefficient profile predicted from the 1D analysis performed by DEEPSOIL code. The following steps show the procedure of specify the Reyleigh damping coefficient that used in 2D nonlinear analyses.

a- Reyleigh damping coefficient (α_R and β_R) over the thickness soil layer are predicted from 1D analysis. The details of this step are given hereinafter.

b- The previous Reyleigh damping coefficients is used as initial profiles in the nonlinear analysis.

c- New profiles for Reyleigh damping coefficients are predicted from step b depend on the level of shear strain and the frequency interval (f_m,f_n)

d- Step c is repeated (two to three times) until reach to the constant values α_R, β_R profiles

Different possible procedure were proposed in the literature to identify the frequency interval (f_m, f_n) [25,26]. Amorosi et al. [13] presented a new procedure to specify frequency interval in order to obtain a better match between the linear time domain and frequency domain analyses whereas, the first natural frequency (f_1) is selected as f_m. The value of f_n should be selected equal to the frequency where the amplification function gets lower than unity. In the current study this procedure is used to obtain the Reyleigh damping coefficient that used in the first trial of nonlinear analysis.
For example, for the case of soft clay deposit exited by Motion 2 earthquake, Fig.7 shows an example of the amplification function of the seismic signal at 15m depth. It shows the frequency interval f_m=0.83Hz at the first peak of amplification function and f_n= 1.25 Hz. The damping ratio at this depth depending on the maximum shear strain is 8%. From Eq. 4 the corresponding Reyleigh damping coefficient are (α_R=0.5014 and β_R=0.0122). In order to construct the α_R and β_R profiles according to Eq. 4 the values of f_m, f_n, maximum shear strain (γ_{max}), Damping ratio depend on γ_{max} should be obtained at different depths along the stratum. Fig. 8 shows the initial profiles of Reyleigh damping coefficient adopted in the PLAXIS for the studied cases.

6. Peak ground acceleration profile and response spectra

The previous calibrated model is used here to predict the peak ground acceleration profiles (PGA) and response spectra for different depths using 2D nonlinear analysis compared to those results obtained with 1D equivalent linear analysis. The comparison of PGA profiles with DEEPSOIL and with PLAXIS analysis is illustrated in Fig. 9 for all studied cases. A good agreement can be observed for deep depths than those obtained for shallow depths. It can be noticed that the results of PGA obtained by nonlinear analysis lower than those observed with equivalent linear analysis especially for stiff clay and strong ground motion.
Fig. 10 shows the comparison between the results of the nonlinear analyses and the corresponding equivalent linear analysis in terms of response spectra computed at different ground motions and different depths along the deposit. The results of nonlinear analyses show a reduction of the spectra as compared to the equivalent linear analysis. This is more pronounced in the shallow depths, between 0 and 10m depth.

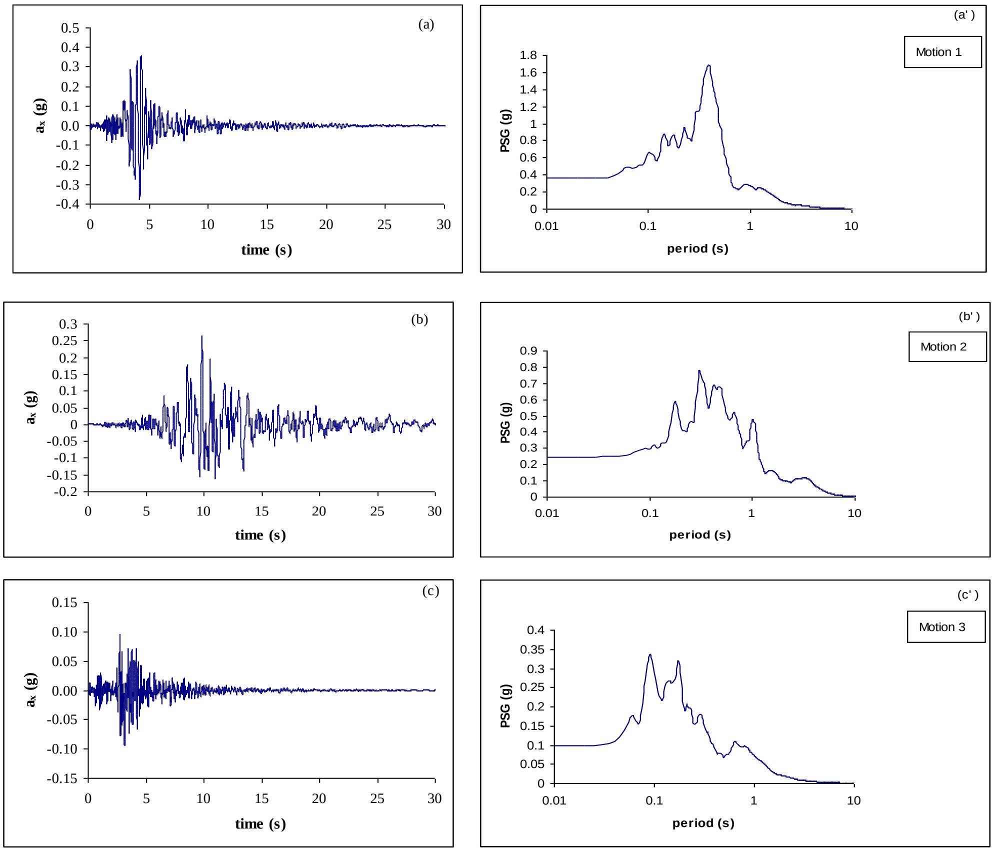

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Fig. 4. Acceleration and response spectrum at to the bed rock of the soft clay in 2D analysis: (a)(a') Motion 1, (b)(b') Motion 2, (c)(c') Motion 3.

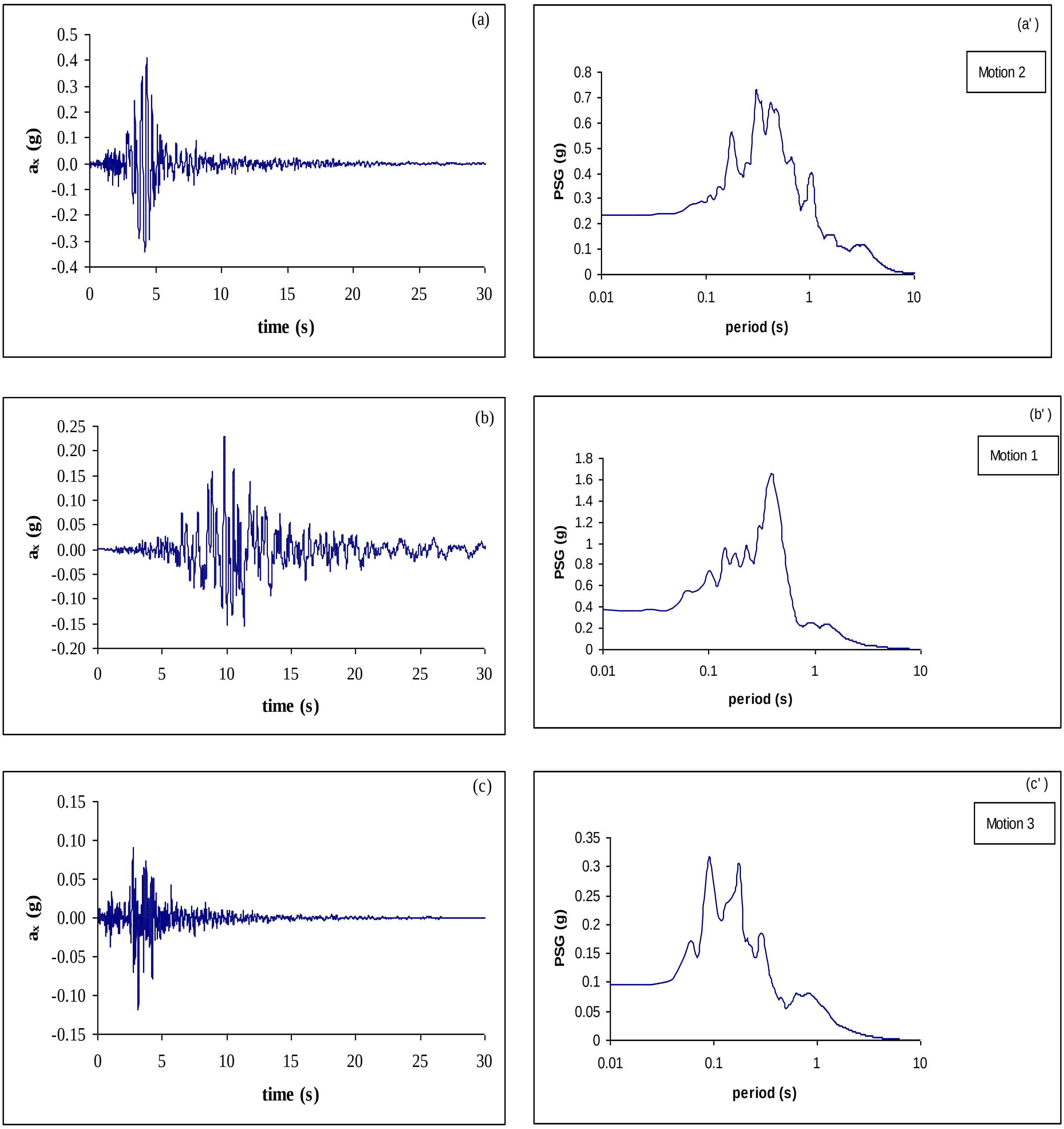


Fig. 5. Acceleration and response spectrum at the bed rock of the stiff clay in 2D analysis: (a) (a') Motion 1, (b) (b') Motion 2, (c) (c') Motion 3.

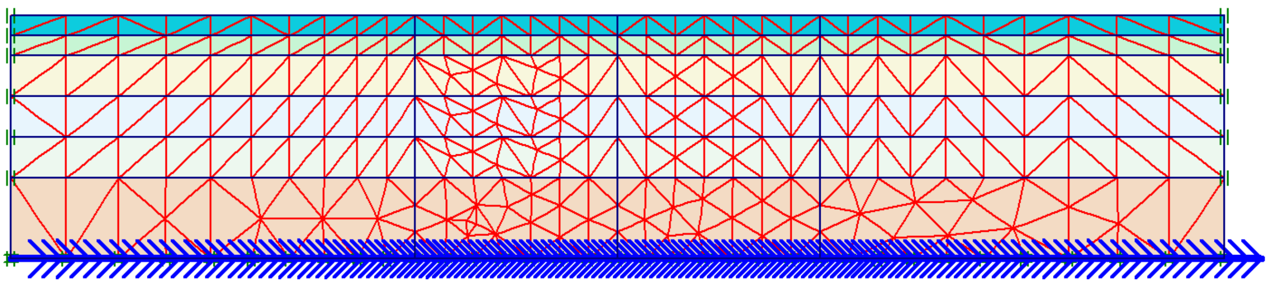


Fig. 6. Sketch of the two dimensional finite element model

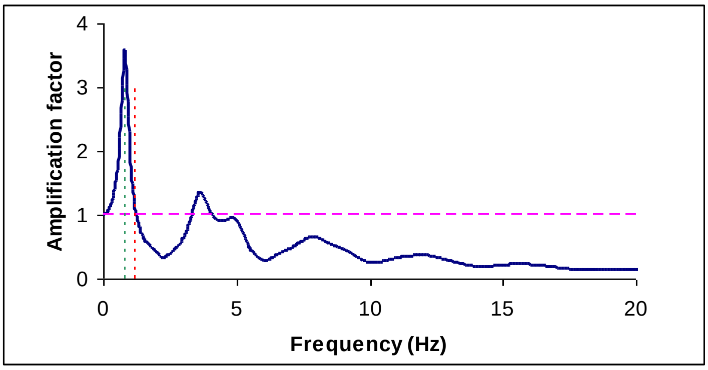


Fig. 7 Amplification function at 15m depth for soft clay deposit and Motion 2 earthquake

7. Effect of peak ground acceleration on shear strain

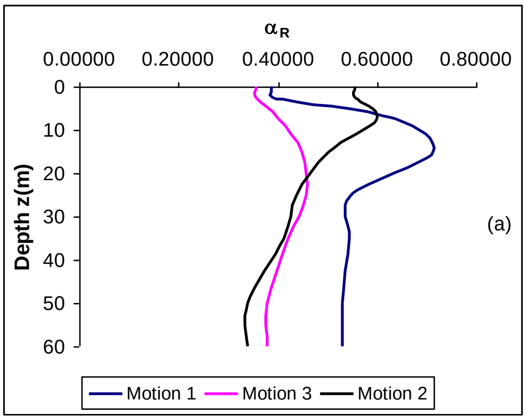
It is well known that the induced shear strain in soil depends on maximum peak ground acceleration in seismic signal. In this section the nonlinear analysis performed with PLAXIS code is used to investigate the effect of peak ground acceleration on the maximum shear strain during ground shaking and the permanent shear strain after the duration of earthquake.

In order to investigate the relationship between the maximum shear strain in soil and maximum peak ground acceleration, Motion 2 earthquake is selected in the current study and scaled to (0.4, 0.8, 1, 1.4, 1.8, 2).

Figure 11 shows an example of the variation of shear strain with time during ground shaking for soft clay deposit at 10 depth and it can be noted that the permanent shear strain in this case is 3e-2%.

The group of curves presented in Fig. 12 shows the effect of peak acceleration on the maximum shear strain at different depths for stiff clay deposit. It can be observed that the shear strain increases linearly with increase of peak ground acceleration. For soft clay, it can be noted that the rate of maximum shear strain increases as the peak ground acceleration increase as shown in Fig. 13.

Fig. 14 shows the relationship between the permanent shear strain and peak ground acceleration for stiff deposit. It shows that the permanent shear strain increases with increase of peak ground acceleration and the relation can be considered linear. For soft clay, the same trend can be noticed and the rate of increase of the permanent shear strain increases with the increase of peak ground acceleration as shown in Fig. 15. Because the plasticity can be obtained in the nonlinear analysis (plastic analysis) permanent displacement and corresponding variation of the effective stress state occur, significantly modifying the soil–structure interaction in any geotechnical context e.g. [5]. Therefore the results obtained by the equivalent linear analysis should not be considered as a right way to modeling strong motion earthquakes especially for soft clay deposit because the nonlinear analysis does not account for the change in soil properties during the of ground motion.



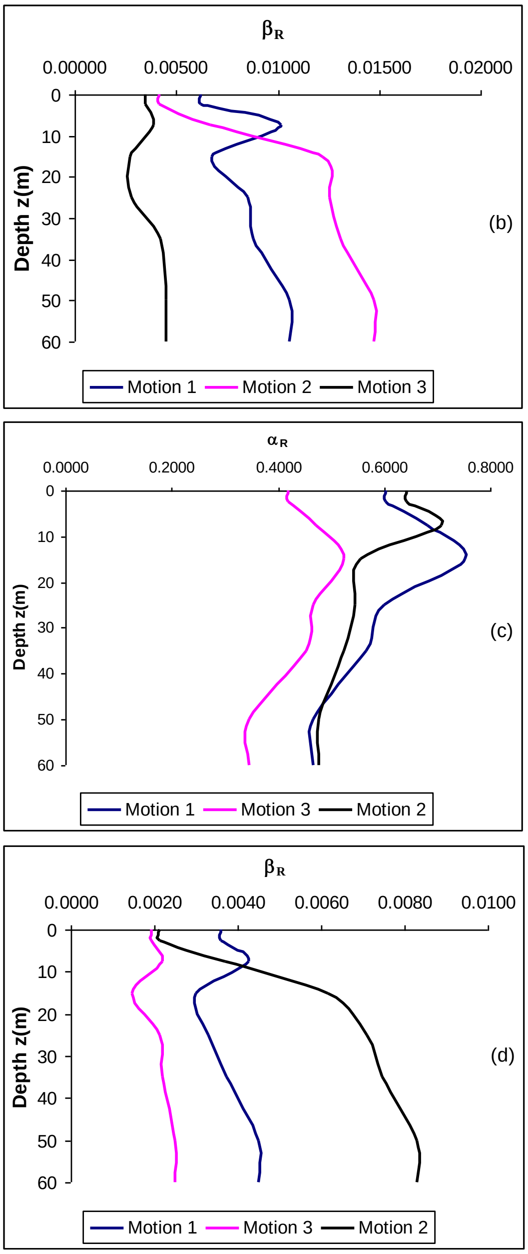
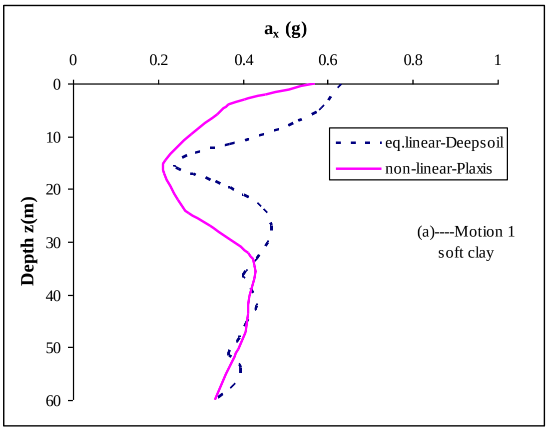
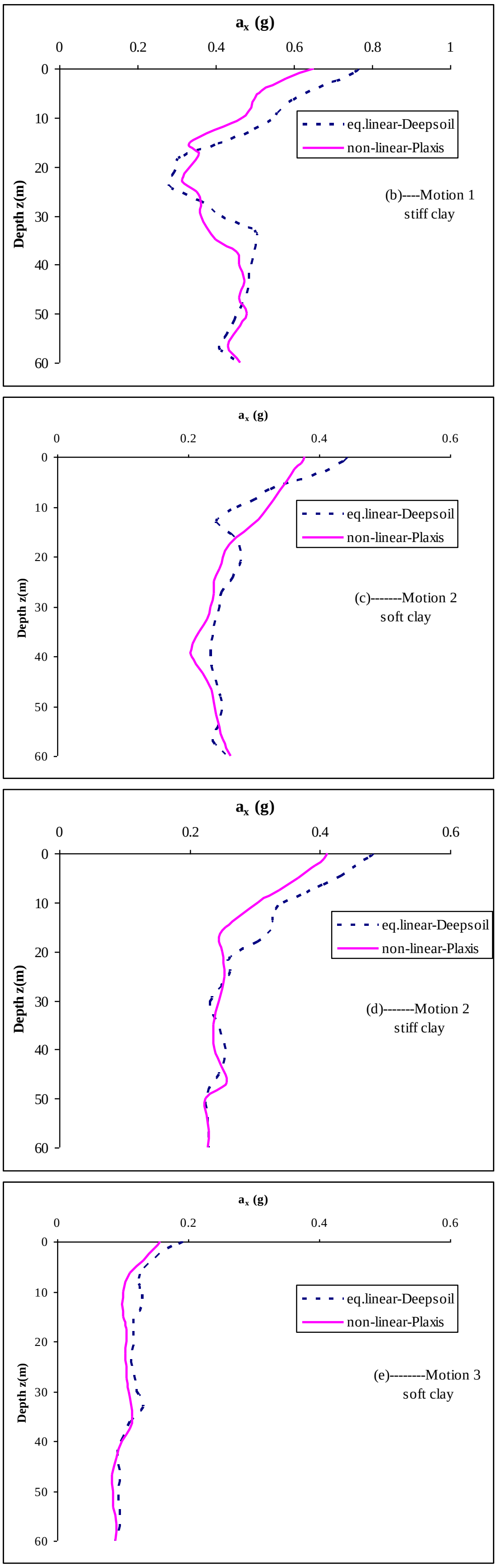


Fig. 8. α_R and β_R profiles (a) α_R for soft clay; (b) β_R for soft clay; (c) α_R for stiff clay; (d) β_R for stiff clay





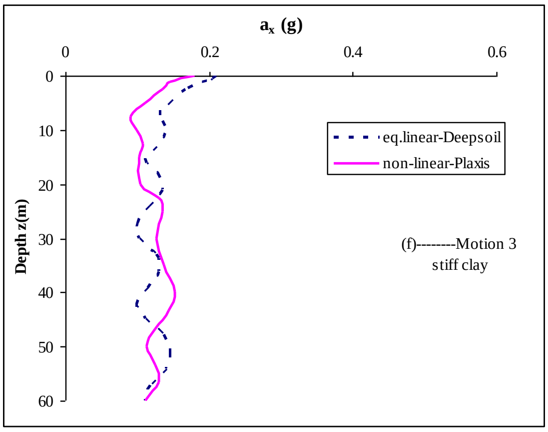
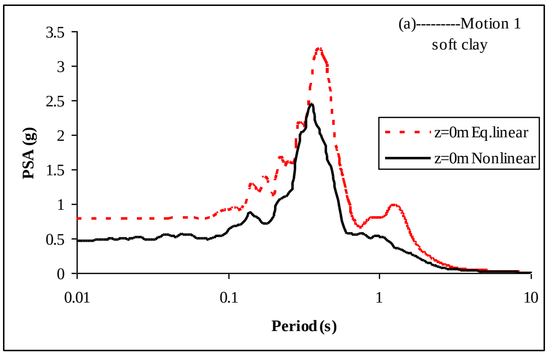
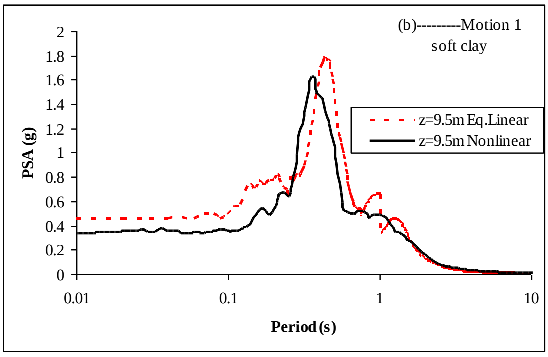
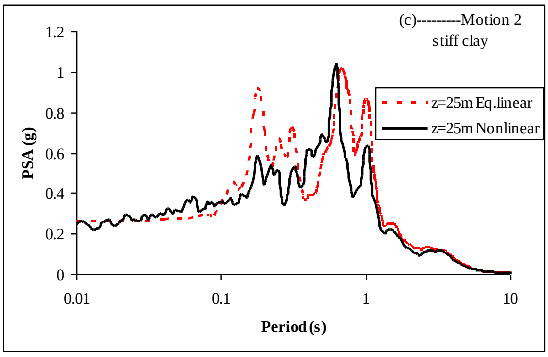
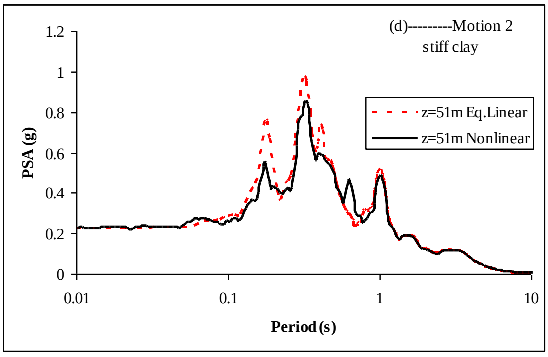


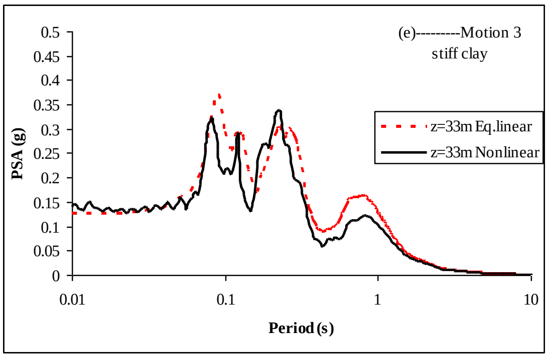
Fig. 9. Profiles of Peak ground acceleration for the studied cases.











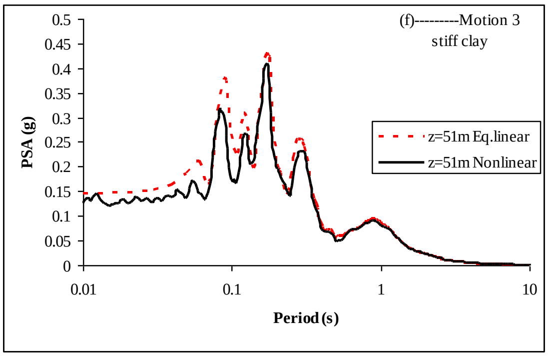


Fig. 10. Comparison between response spectra obtained with 1D equivalent linear analysis and 2D nonlinear analysis

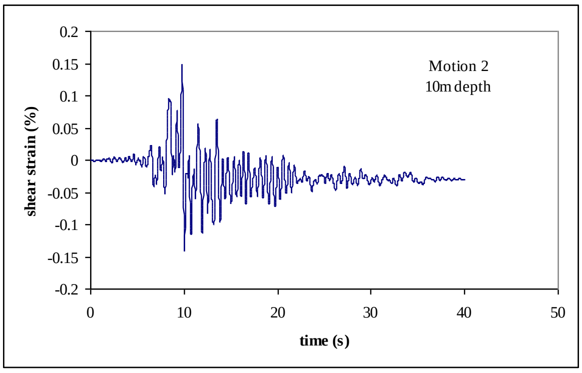


Fig. 11.Variation of shear strain with time for soft clay deposit and 10m depth.

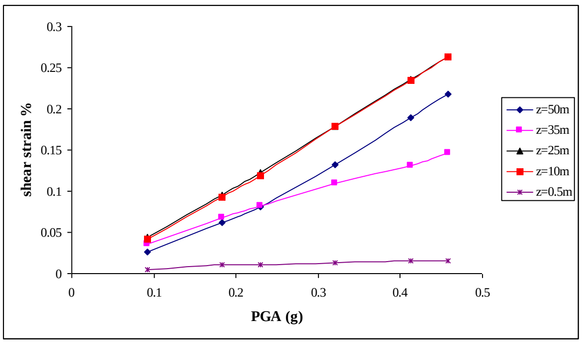


Fig. (12) Effect of peak ground acceleration on the maximum shear strain for stiff clay deposit.

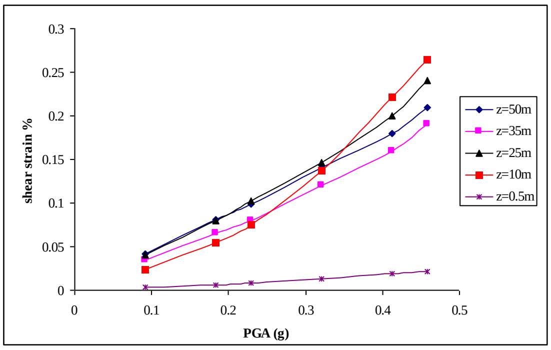


Fig. 13. Effect of peak ground acceleration on the maximum shear strain for soft clay deposit.

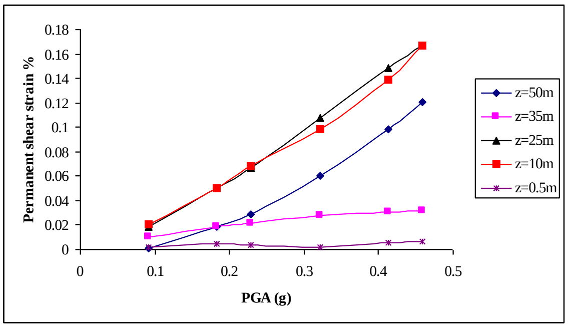


Fig. 14. Effect of peak ground acceleration on the permanent shear strain for stiff clay deposit.

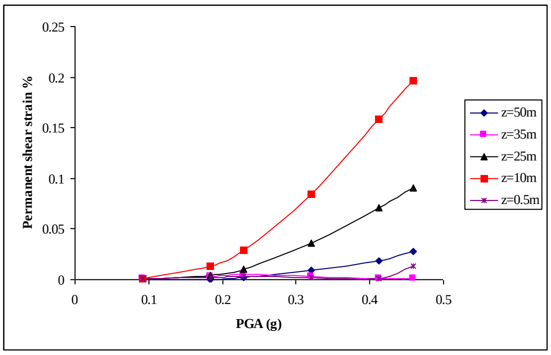


Fig. 15. Effect of peak ground acceleration on the permanent shear strain for soft clay deposit.

8. Conclusions

In this paper a set of nonlinear 2D dimensional finite element analyses were performed to describe the nonlinear behavior of soil deposits during and after ground shaking. The stiffness values and the amount of viscose damping are investigated in equivalent linear analysis using 1D analysis to calibrate the plastic analysis models using 2D analysis. The comparison between nonlinear and equivalent linear analysis for three different acceleration time histories was also presented. The effect of peak ground acceleration on the maximum and permanent shear strain was investigated. Almost results showed a contraction of peak ground acceleration profile and the spectra as compared to the equivalent linear analysis especially in the uppermost portion of the deposit. For lowermost portion a good agreement between the results obtained by equivalent linear solution and those obtained by the 2D nonlinear solution was observed.

The maximum and permanent shear strain induced in the soil increases as increase of peak ground acceleration. The rate of increase in shear strain increases as increase of peak ground acceleration for soft soil and linearly for stiff clay therefore, the results obtained by the equivalent linear analysis should not be considered as a right way to modeling strong motion earthquakes especially for soft clay deposit because the nonlinear analysis does not account for the change in soil properties during the of ground motion.

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