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A COMPARATIVE STUDY OF ELASTIC AND NONLINEAR SOIL RESPONSE ANALYSIS

Anna KARATZETZOU[1], Stavroula FOTOPOULOU[2], Evi RIGA[3], Sotiria KARAPETROU[4], Grigoris TSINIDIS[5], Evangelia GARINI[6], Kyriazis PITILAKIS[7], Nikos GEROLYMOS[8] and George GAZETAS[9]

ABSTRACT

Earthquake case histories of soil response over the last 70 years reveal the strongly non-linear, hysteretic behaviour of soils. Over the years, numerous efforts have been made to model the non-linear behaviour of soils during earthquakes. The paper presents a comprehensive comparative study, performed in the framework of PRENOLIN project (https://www-cashima.cea.fr/), of the elastic and non-linear response of two idealised soil profiles: a single layer soil deposit of 20 m depth with constant shear wave velocity and an soil profile of 100 m depth with a gradient (parabolic) distribution of shear wave velocity with depth. The profiles are subjected to a number of idealised and recorded time- history accelerations.

The numerical simulations are performed with the research codes NL-DYAS, DEEPSOIL and OPENSEES as well as the commercially available finite element programme ABAQUS, incorporating several appropriate constitutive models. Results are presented in terms of acceleration time histories, shear stresses and strains. The elastic and non-linear response computed with the aforementioned four codes is compared with each other. Similarities and differences between the programs in predicting the seismic response are highlighted.

INTRODUCTION

One of the crucial issues in earthquake engineering practice is the proper evaluation of the ground motion. In the simplest case, this evaluation is performed through 1D site response analysis, implementing the equivalent linear approximation (Schnabel et al., 1972) to model the soil dynamic response. Although the method is cost-effective compared to more detailed non-linear dynamic analysis, it does not account for several characteristics of ground motion in case of strong earthquakes including the computation of permanent ground deformations. Actually, non-linear soil behaviour is observed in all recent large earthquakes worldwide affecting seriously the site-specific ground motion.

[1] Ph-D Candidate, AUTH, Thessaloniki, Greece, akaratze@civil.auth.gr
[2] Post-Doctoral Researcher, AUTH, Thessaloniki, Greece, sfotopou@civil.auth.gr
[3] Ph-D Candidate, AUTH, Thessaloniki, Greece, eviriga@civil.auth.gr
[4] Ph-D Candidate, AUTH, Thessaloniki, Greece, gkarapet@civil.auth.gr
[5] Ph-D Candidate, AUTH, Thessaloniki, Greece, gtsinidi@civil.auth.gr
[6] Post-Doctoral Researcher, NTUA, Athens, Greece, geocvemp@yahoo.gr
[7] Professor, AUTH, Thessaloniki, Greece, pitilakis@civil.auth.gr
[8] Assistant Professor, NTUA, Athens, Greece, gerolymos@gmail.com
[9] Professor, NTUA, Athens, Greece, gazetas@ath.forthnet.gr

It is therefore important to use adequate numerical tools that can describe efficiently and realistically the soil non-linearity when predicting site response.

Several commercial or research numerical codes which are currently available, are capable of performing non-linear soil response analysis. The analysis is commonly performed in the time domain with the soil non-linear response being modelled with a non-linear constitutive model of various complexity, ranging from a simple elastic-perfectly plastic model (e.g. a Mohr-Coulomb model) to a more complicated and rigorous model (e.g. models that account for large strains, liquefaction etc). In this paper we discuss representative results from a series of numerical analyses with four different numerical codes on two idealized soil profiles. The objective of the analyses, which were performed assuming either visco-elastic or elasto-plastic soil response is to cross compare the results of each code in terms of accelerations, strains and stresses and through this comparison to qualitatively evaluate the accuracy of each model. To extend our comparisons, the analyses are performed under several assumptions regarding soil stratigraphy and properties, input motion characteristics, and the soil-bedrock interface (elastic and rigid bedrock). The results are discussed in terms of acceleration time histories, transfer functions, distributions of peak horizontal acceleration and shear strain and stress-strain loops.

DESCRIPTION OF CASE STUDIES

A series of elastic, visco-elastic and non-linear analyses are performed using two "canonical" soil profiles P1, and P2, which represent ideal site conditions (Fig. 1). Soil profile P1 (Fig.1a) represents a homogeneous single layer soil deposit of 20m in depth with constant shear wave velocity V_s=300m/s. The shear wave velocity of the underlying bedrock $V_{s,rock}$ is equal to 1000m/s. Soil profile P2 (Fig.1b) resembles an inhomogeneous soil profile of 100m in depth with gradient shear wave velocity V_s, varying with depth z according to the following generic form:

$$V_s = V_{s1} + (V_{s2} - V_{s1}) \cdot [(z - z_1) / (z_2 - z_1)]^a \tag{1}$$

where z_1:depth at top of deposit (=0m), z_2:depth at bottom of deposit (=100m), V_{s1}: V_s at top of deposit (=150m/s), V_{s2}: V_s at bottom of deposit (=500m/s), α: a constant (=0.25). The shear wave velocity of the underlying bedrock $V_{s,rock}$ is equal to 2000m/s.

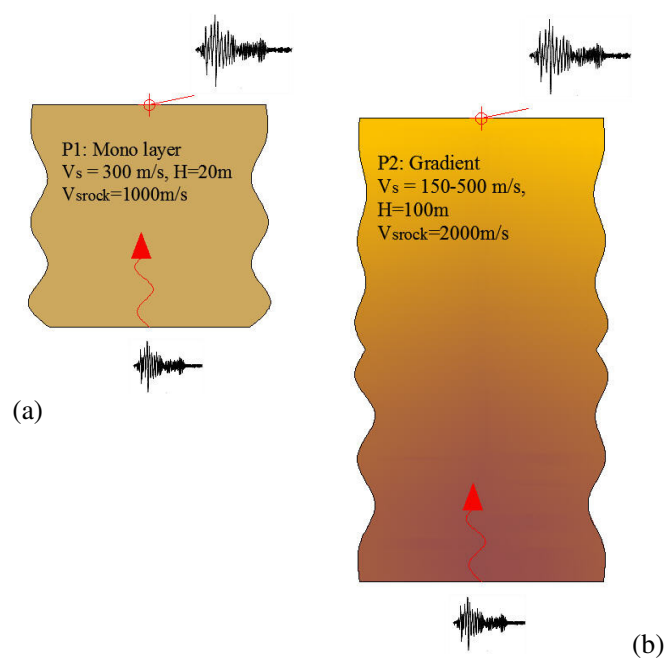


Figure 1. (a) Profile P1, (b) Profile P2

Compression wave velocity V_p is calculated as follows:

$$V_p = \sqrt{2V_s^2\left(\frac{\nu-1}{2\nu-1}\right)} \tag{2}$$

where ν is the Poisson coefficient, assumed to be equal to 0.4 for sediments and 0.3 for rock. Mass density ρ is assumed to be equal to 2000kg/m^3 for sediments and 2500 kg/m^3 for rock.
Attenuation for elastic and visco-elastic analyses is characterized by a quality factor Q, which is considered as follows:
(i) elastic analyses: Q=5000 for both sediments and rock
(ii) visco-elastic analyses: $Q_{sed}=V_s/10$ for sediments and $Q_{rock}=200$ for rock.
The corresponding attenuation is described in terms of viscous damping that is modeled in the frequency depended Rayleigh type in ABAQUS and OPENSEES, while in DEEPSOIL and NL-DYAS is assumed constant.

The elastic properties of the profiles are summarized in Table 1.In all cases, the analyses are performed assuming dry conditions. To this end, the analyses are performed in total stresses.

Table 1. Elastic properties of the soil profiles

Soil profile	z (m)	Vs (m/s)	Vp (m/s)	ρ (kg/m^3)	Q elastic	Q viscoelastic
P1	0-20	300	700	2000	5000	30
	bedrock	1000	1900	2500	5000	200
P2	0-100	150-500	360-1220	2000	5000	15-50
	bedrock	2000	3700	2500	5000	200

The degradation of the soil stiffness and the damping increase with increased shear strains are described with proper G-γ-D curves. More specifically, shear modulus reduction G/G_{max} and damping ratio ξ curves are computed by equations (3) and (4):

$$G/G_{\max} = 1/(1+\gamma/\gamma_{ref}) \tag{3}$$

$$\xi = \xi_{\min} + (\xi_{\max} - \xi_{\min})\cdot(\gamma/\gamma_{ref})/(1+\gamma/\gamma_{ref}) \tag{4}$$

where

$$\gamma_{ref} = \tau_{\max}/G_{\max} \tag{5}$$

$$\tau_{\max} = \sigma' \sin\varphi \tag{6}$$

$$\sigma' = \sigma'_v(1+2K_0)/3 \tag{7}$$

$$\sigma'_v = \rho\cdot g\cdot z \tag{8}$$

$$g = 9.81 \text{ m/s}^2 \tag{9}$$

$$K_0 = (1-\sin\varphi)\cdot OCR^{\sin\varphi} \tag{10}$$

$$\varphi = 30^o \tag{11}$$

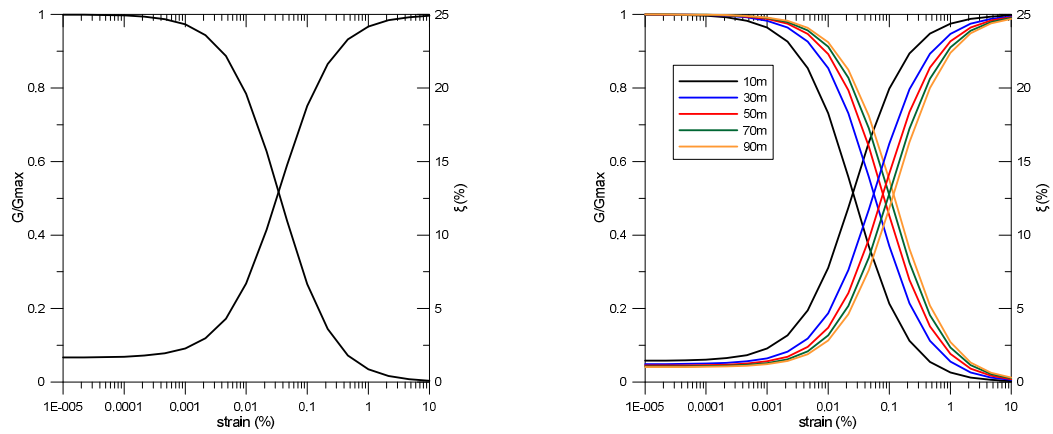
$$OCR = 1 \tag{12}$$

$$G_{\max} = \rho\cdot V_s^2 \tag{13}$$

$$\xi_{\min} = 1/(2Q_s) \tag{14}$$

$$\xi_{\max} = 0.25 \tag{15}$$

The resulting shear modulus reduction (G/G_{max}) and damping ratio (ξ %) curves for soil profile P1 are illustrated in Fig.2a. For soil profile P2, five sets of shear modulus reduction and damping curves are calculated for depth intervals of 20m, referenced at 10, 30, 50, 70, and 90m depth (Fig.2b). In addition to the hysteretic damping introduced by each non-linear constitutive model, a small amount of viscous damping is assigned to account for the energy dissipation during the elastic part of the cyclic response.

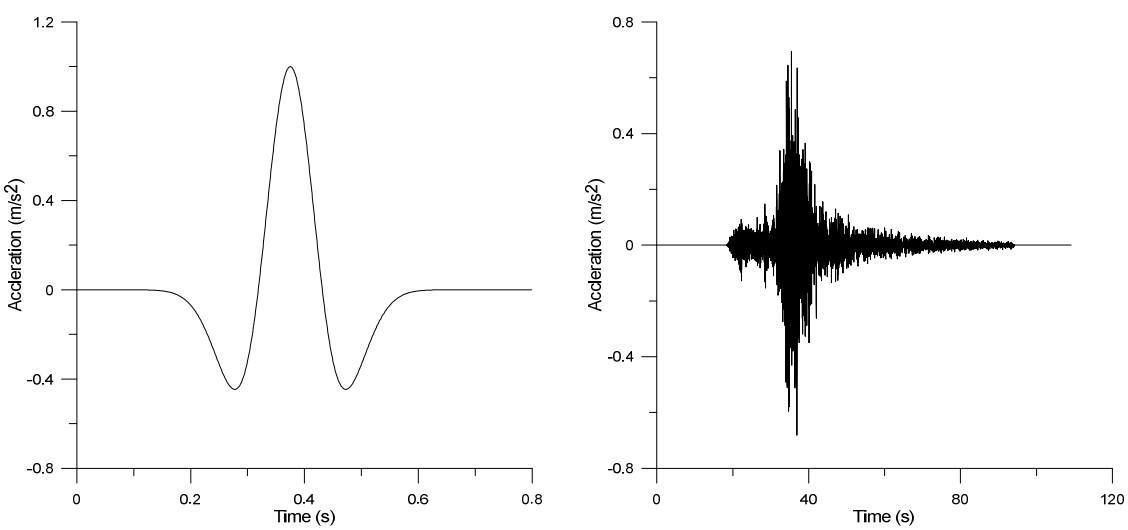


(a) (b)

Figure 2. Soil reduction G/G_{max} and damping ξ(%) curves for soil profiles (a) P1 and (b) P2.

Analyses are performed assuming constant shear strength for depth intervals of 20m. This strength is estimated at the middle of each soil layer assuming a friction angle $\varphi = 30^o$. Considering the increase of the mean effective stress with depth, soil shear strength increases with depth.

The analyses are carried out using both an impulse and a real accelerogram as input motion at the model base. For the elastic and visco-elastic analyses we used two input motions: a Ricker type pulse with corner frequency T_c=4Hz (Fig.3a) and the E-W component of a real accelerogram recorded at station IWTH17 (V_{s30}= 1200m/s) of the Kik-Net array (Fig.3b). The non-linear analyses were performed with the same accelerogram used for the elastic analyses, scaled at Peak Ground Acceleration PGA equal to 0.5, 2.0 and 5.0 m/s^2.



(a) (b)

Figure 3. Input motions used for the elastic and visco-elastic analyses: (a) Ricker pulse, (b) real accelerogram

In order to study the effectiveness of numerical codes to model either rigid or flexible substratum, the analyses are performed for different boundary conditions at the sediment/bedrock interface.

In the ensuing, results will be illustrated in figures with the following code name: P[profile_no]_[damping]_[input]_Z[depth]_[bedrock], where:

profile no: can be 1 or 2 depending on the soil profile number
damping: can be 'el' or 've' for elastic and visco-elastic case respectively
input: can be 'puls' or 'real' for pulse like signal and for real accelerogram respectively
depth: can take values 0,1,2..10 for the accelerations and 0,1,...9 for the stresses and strains. When the depth value is equal to zero, the acceleration time history is at free field conditions, while the stress-strain values are at depth equal to H/20, where H the soil profile depth.
bedrock: can be 'E' or 'R' for elastic halfspace and rigid bedrock respectively.

GROUND RESPONSE MODELING

Elastic, visco-elastic and non-linear ground response analyses are examined utilizing different soil models. The important issues involved in the modelling of a soil profile subjected to earthquake loading when the former behaves in the elastic, visco-elastic or non-linear range, are highlighted using four different commercial and research numerical codes, namely: ABAQUS (ABAQUS, 2012), OPENSEES (Mazzoni et al., 2009), DEEPSOIL (Hashash and Park 2001, 2002; Park and Hashash 2004; www.uiuc.edu/~deepsoil) and NL-DYAS (Gerolymos and Gazetas, 2005, Drosos et al., 2012). ABAQUS and OPENSEES are generic finite element codes, while DEEPSOIL and NL-DYAS are 1D soil response analysis programs. In all cases, soil deposits are discretized to account for the efficient reproduction of the waveforms of the whole frequency range under study.

In ABAQUS and OPENSEES, the rigid base is modeled in simplified way by constraining vertical displacements, while elastic base is modeled using the Lysmer and Kuhlemeyer (1969) scheme, introducing proper dashpots. Time discretization is an important element for the analysis accuracy. All codes offer an automatic time incrementation scheme, according to which the time step of the analysis is properly selected and changed to achieve a stable solution. All codes will be examined for the simplest case of 1D shaking, even if some of these codes (e.g. OPENSEES) are capable of performing multi-dimensional earthquake loading ground response analyses.

A kinematic hardening model combined with a Von-Mises failure criterion and an associated plastic flow rule is used to model soil non-linear response under ground shaking in ABAQUS (Anastasopoulos et al, 2011). The model parameters are calibrated based on the given G-γ-D curves and the strength of the soil stratum. The analyses are performed with an implicit algorithm scheme, using full Rayleigh viscous damping formulation and Masing rules to calculate the hysteretic damping according to Masing unloading/reloading criteria. Full Rayleigh viscous damping as well as Masing criteria is also used in OPENSEES. In OPENSEES we utilized the multi-yield surface plasticity model with an associative flow rule as described in Ragheb, 1994, Parra, 1996, Yang, 2000. In DEEPSOIL we used the embedded MRDF pressure-dependent hyperbolic model with non-Masing criteria, which introduces a reduction factor into the hyperbolic model by Hashash and Park (2001), also available in the code. DEEPSOIL provides a frequency independent damping formulation, as well as three types of Rayleigh damping (simplified, full and extended, with one, two and four modes needed to define viscous damping respectively). The recommended frequency independent damping formulation was used for the analyses. NL-DYAS uses the smooth hysteresis model originally proposed by Bouc 1971 and Wen 1976, which was later extended by Gerolymos and Gazetas (2005), Drosos et al. (2012).

All models were properly calibrated using the aforementioned G-γ-D curves. More specifically, the targeted G-γ-D curves (Fig. 2) were fitted either automatically (e.g. OPENSEES, DEEPSOIL) or manually. In ABAQUS this was achieved by comparing the numerically estimated G-γ-D curves, computed by simulations of cyclic simple shear tests for different levels of shear strains (Gerolymos and Gazetas, 2006, Anastasopoulos et al., 2011) with the target curves (Fig.2).

Based on the characteristics of the utilized codes summarized in Table 3, it is normally expected that ABAQUS and OPENSEES will give more or less similar results as both of them use implicit / FEM numerical scheme, full Rayleigh damping and Masing rules. NL-DYAS on the other hand uses quite different assumptions compared to the other three codes, as it uses an explicit finite difference solver. Finally, in DEEPSOIL and NL-DYAS a constant viscous damping and no Masing rules are utilized.

Table 2. Main characteristic of the utilized codes

Code	**Nonlinear model**	**Constitutive Model**	**Numerical Scheme**	**Rayleigh Damping**	**Hardening rule**
ABAQUS	Armstrong and Frederick (1966).	Single-yield surface plasticity (Gerolymos and Gazetas, 2006, Anastasopoulos et al, 2011)	Implicit / FEM	Full	No (nonlinear Kinematic)
OPENSEES	Mazzoni et al. (2009) 2009	Multi-yield surface plasticity (Ragheb, 1994; Parra, 1996; Yang, 2000)	Implicit / FEM	Full	Yes (Masing)
DEEPSOIL	Hashash and Park (2001, 2002)	MRDF pressure-dependent hyperbolic model Hashash et al. (2012)	Newmark (1959) β method	No (frequency independent damping)	No
NL-DYAS	Gerolymos and Gazetas (2005), Drosos et al. (2012)	Smooth hysteresis model originally proposed by Bouc (1971) and Wen (1976). Extended by Gerolymos and Gazetas (2005)	Explicit Finite Difference Solver	No (viscous damping)	No

ELASTIC SOIL RESPONSE

Representative results of the soil response under purely elastic and visco-elastic behaviour using the four numerical models are presented and discussed in this section. Fig.4 shows typical acceleration time histories at the ground surface considering purely elastic response of the homogeneous soil profile P1. The viscous damping is about 1% and the bedrock is simulated as elastic half-space using appropriate dashpot values. Fig.4a illustrates the whole time history (15s-80s), while, to better illustrate the differences, in Fig.4b a time window of the time-histories is presented referring to the maximum computed acceleration (34s-36s). Fig.5a compares the computed horizontal accelerations (time window from 34s to 36s), for profile P1 over elastic half-space assuming visco-elastic soil response and the real accelerogram as input. Similar comparison is made in Fig.5b where the surface horizontal acceleration is presented for the soil profile subjected to a pulse signal and rigid bedrock conditions.

It is observed that for both purely elastic and visco-elastic analyses all implemented numerical codes predict quite similar responses. Numerical results are in good agreement with each other both in

terms of acceleration amplitudes and frequencies. Even for the low damping value (Fig.4) results in terms of acceleration amplitudes almost coincide between the different codes.

Some numerical problems were detected in case of almost purely elastic analyses when rigid bedrock is assumed. The unrealistically small damping considered in this case, resulted in small attenuation of the computed acceleration after main shaking, in some codes. The problem was less evident in the case of the elastic base, as part of the seismic energy was absorbed by the dashpots at the model base.

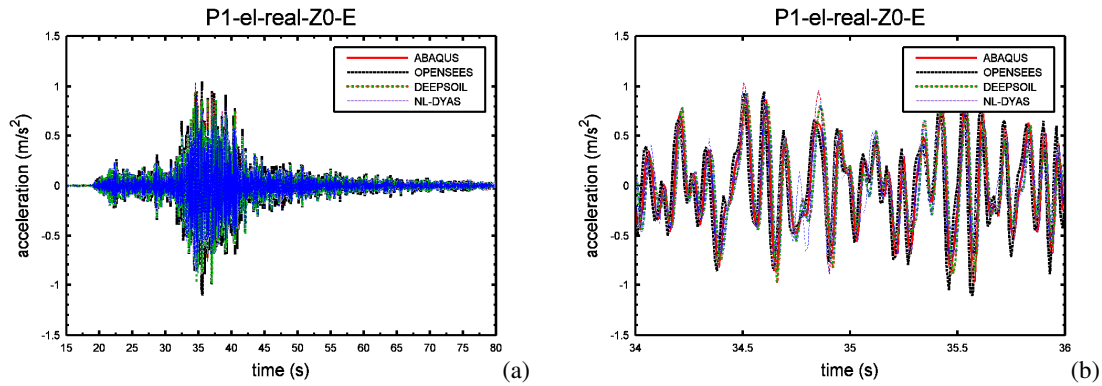


Figure 4. Acceleration time histories at the ground level for elastic response of soil profile P1, elastic case for damping, real record as input and elastic half-space (a) 15s-80s, (b) 34s-35s

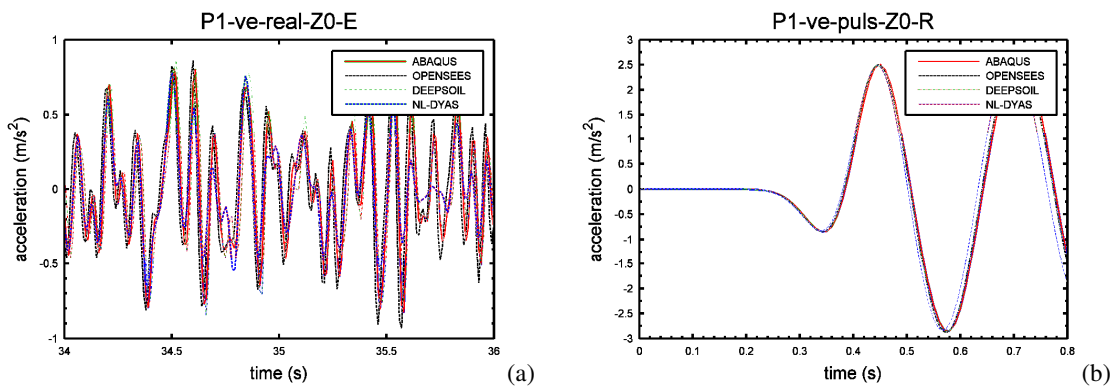


Figure 5. Acceleration time histories at the ground level for profile P1 elastic soil response, visco-elastic case for damping, (a) real record as input and elastic half-space and (b) pulse record as input and rigid bedrock

The soil resonant frequencies, derived from transfer functions, were found in good agreement with the theoretical closed form solutions. More specifically, according to these solutions resonant frequencies are estimated equal to 3.75 Hz and 1.16 Hz for profile P1 and profile P2, respectively. Numerical analyses reveal similar resonant frequencies (Fig.6). For profile P1 (Fig.6a) all numerical codes give exactly the same first frequency and amplitude. The second frequency is, however, slightly lower when ABAQUS, OPENSEES and DEEPSOIL are used. This difference is however more noticeable for ABAQUS and OPENSEES, which give about 5% lower value compared to the corresponding theoretical expected frequency (11.25Hz). The second soil frequency is the same as the theoretical one when NL-DYAS code is utilized, even if the amplitude for this frequency is up to 50% lower compared with the other three codes. This can be attributed to the different approximations of the codes concerning the viscous damping simulation.

In case of soil profile P2 (Fig.6b), all codes give almost similar results both in terms of frequency and amplitude at least for the first three resonant frequencies. For higher resonant frequencies ($f > 6.5$Hz) results differ mainly in terms of amplitude but also for frequencies. Again, this should be expected due to the different damping formulation each code uses.

It is important to mention that for the elastic case and for the frequency range of interest in engineering practice ($f<6.5$Hz) all studied codes give similar results.

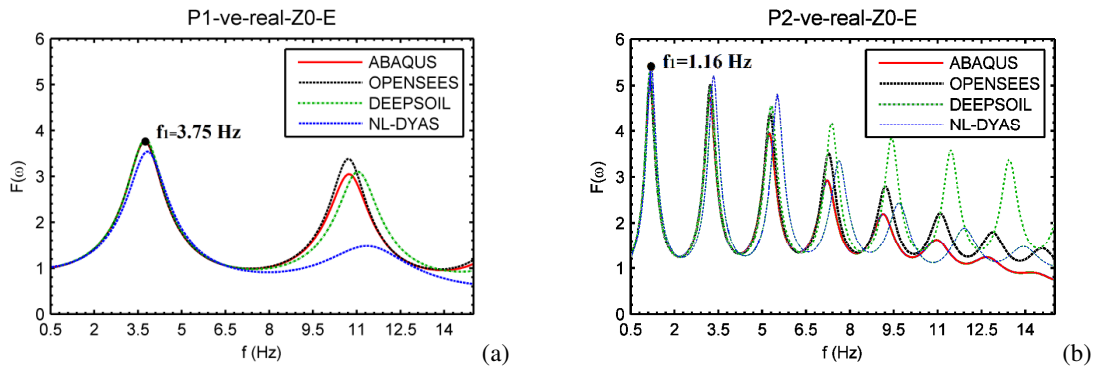


Figure 6. Transfer functions (ground level to bedrock level Fourier) for elastic response of (a) P1 profile, visco-elastic case for damping, real record as input and elastic half-space d) P2 profile, visco-elastic case for damping, real record as input and elastic half-space

Fig.7 depicts the peak horizontal acceleration with depth for profile P1 (Fig.7a) and P2 (Fig.7b). Results refer to visco-elastic analysis using the real ground motion record assuming an elastic base. In case of soil profile P1 results are in good agreement for the first 6 m except for NL-DYAS where the acceleration amplitudes are smaller compared to the three other codes for a depth of 2m from the surface. For depths greater than 6m the differences between ABAQUS, OPENSEES and DEEPSOIL are still minor, while on the other hand NL-DYAS gives smaller accelerations values. This observation is expected considering that in higher frequency values the amplitudes using NL-DYAS were lower (see Fig. 6a); it is also due to the fact that at higher depths the input frequency range is higher. In case of soil profile P2 (Fig.7b), all numerical codes predict in general comparable results with the deviations between the results being in the order of 10% to 30%. At the ground surface of the 20m soil profile P1 differences are not important (Fig.7a), while in case of the gradient profile P2 computed accelerations differ over 35%. In particular, ABAQUS and OPENSEES compute lower acceleration values at ground surface compared to DEEPSOIL and NL-DYAS. This observation may be attributed to the different simulation of soil damping. Indeed, in case of deeper soil profiles Rayleigh damping calibration affects more the computed response.

It is also noticed that ABAQUS and OPENSEES follow the same trend regarding the variation of peak acceleration amplitudes with depth not only for profile P1 but also for P2 where the differences between the various codes are more pronounced. At some depths NL-DYAS seems to follow a slightly different trend compared to the other codes. Considering however the different approximations of each code regarding the viscous damping simulation, the comparison of the peak acceleration values with depth is considered overall satisfactory.

Representative maximum shear strain variation with depth is presented in Fig. 8. Results refer to visco-elastic soil response of soil profiles P1 (Fig.8a) and P2 (Fig.8b), excited with the real ground motion record. Shear strains are almost identical for both profiles with differences being on average 3-15% in all codes.

NONLINEAR SOIL RESPONSE

Fig. 9 presents comparisons of acceleration time histories for soil profile P1 computed at the ground surface when subjected to the real record scaled at peak ground acceleration equal to 5 m/s^2. To study the effect of bedrock modeling (e.g. rigid or elastic bedrock), numerical results, referring to both rigid and elastic half space, are presented. Bedrock simulations have negligible effect on the frequency characteristics of the computed time histories at soil surface. Actually, signals phase is not modified significantly between the two cases, indicating that frequency characteristics are similar irrespectively of the bedrock type. On the other hand, bedrock type seems to have an important effect on the

acceleration amplitude. Indeed, for the case of rigid bedrock, acceleration amplitudes are higher, as theoretically expected.

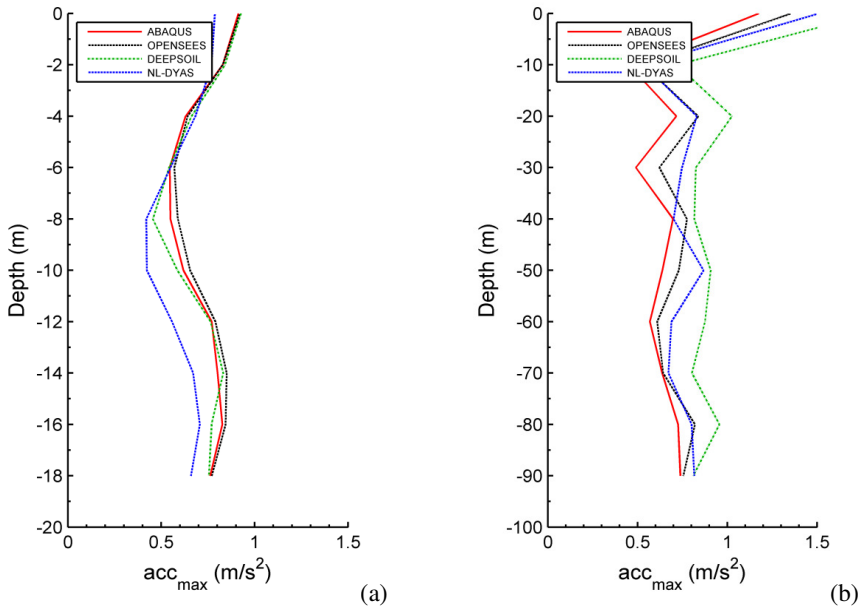


Figure 7. Peak horizontal accelerations with depth for the elastic soil response, visco-elastic case for damping, real record as input and elastic half-space for profiles (a) P1 and (b) P2

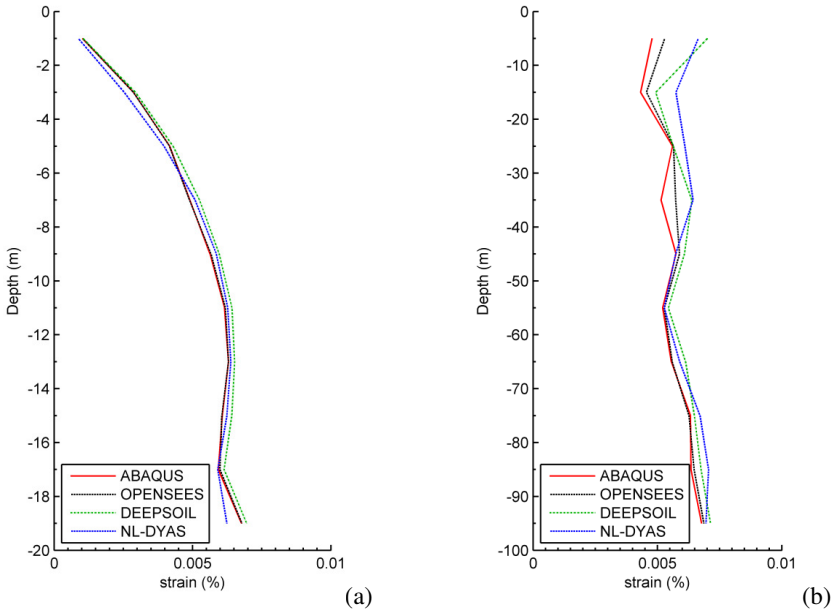
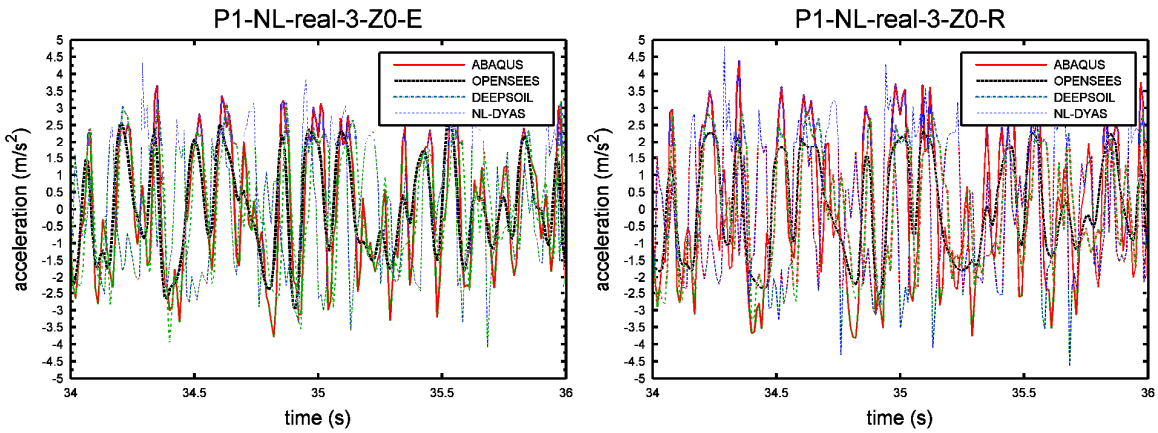


Figure 8. Maximum strain % with depth for the elastic soil response, visco-elastic case for damping, real record as input and elastic half-space for profiles (a) P1 and (b) P2

Comparing different codes it is observed that ABAQUS, OPENSEES and DEEPSOIL result in quite similar response in terms of frequency characteristics (similar phase). On the contrary NL-DYAS results are quite distinct compared to the other codes. This observation indicates that stiffness is more or less similarly captured by ABAQUS, OPENSEES and DEEPSOIL contrary to NL-DYAS.

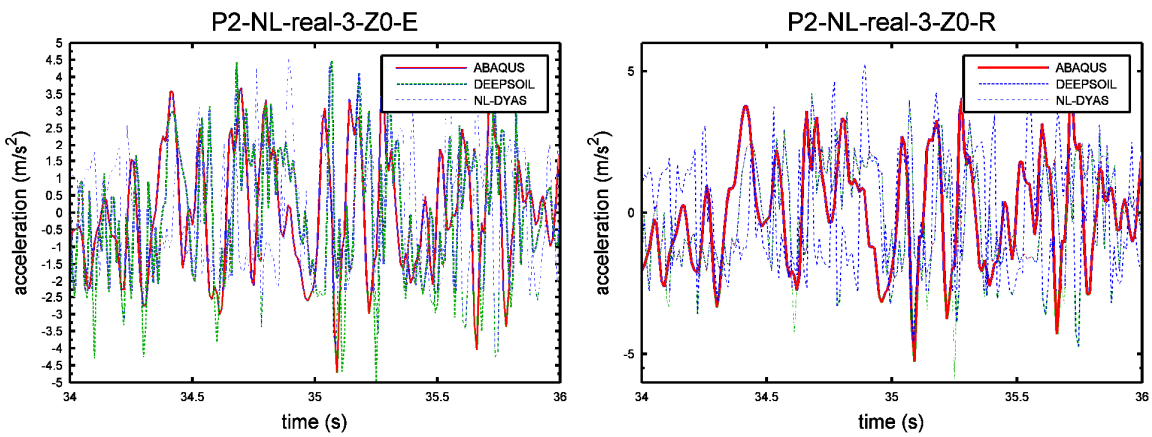
As far as the amplitude is concerned, OPENSEES gives lower values. This implies that the hysteretic damping is over-predicted by the model compared to the other codes, where the amplitudes are similar. Although OPENSEES and ABAQUS are using similar constitutive models the calibration procedures they use (automatic in OPENSEES and manual in ABAQUS), may be responsible for the observed differences at the final computed response.

In case of soil profile P2 (Fig.10) the amplitudes of peak horizontal accelerations are again in relatively good agreement between the different methods. However, the phases are quite distinct indicating discrepancies in estimated stiffness of the soil deposits in different numerical codes. Considering the high amplitude of the seismic input motion (0.5g at 100 m depth), the soil gradient model and the way that this is modeled in each code, and differences on the way the non-linear soil response is accounted for in each code, we may conclude that these rather limited differences should be expected and they are in general within the engineering accuracy margin.



(a) (b)

Figure 9. Acceleration time history (peak acceleration windows) at the ground level for profile P1 and non-linear soil response for (a) elastic half-space and (b) rigid bedrock



(a) (b)

Figure 10. Acceleration time history windows at the ground level for profile P2 non-linear soil response and (a) elastic half-space and (b) rigid bedrock

Further conclusions can be drawn from Fig.11 where the maximum shear stresses, shear strains and accelerations distributions along with depth are presented. Results concern profile P1 on a elastic half space. Considering the different hypotheses used by each code to account for soil non-linearity, the results are again satisfactory. More specifically, the maximum differences are not exceeding 20% for stresses at depth of 19m while this difference is further decreased towards the surface. In terms of

strains the results are generally in good agreement and the maximum difference, again at 19m, don't exceed 50%. It is noticed however that this large difference concerns only this depth and only DEEPSOIL and thus the results are considered satisfactory. Concerning the peak acceleration amplitudes, the differences are rather important among the four codes; OPENSEES give the lower values and NL-DYAL the highest at almost all depths (almost double compared to OPENSEES). ABAQUS and DEEPSOIL give comparable results in all cases.

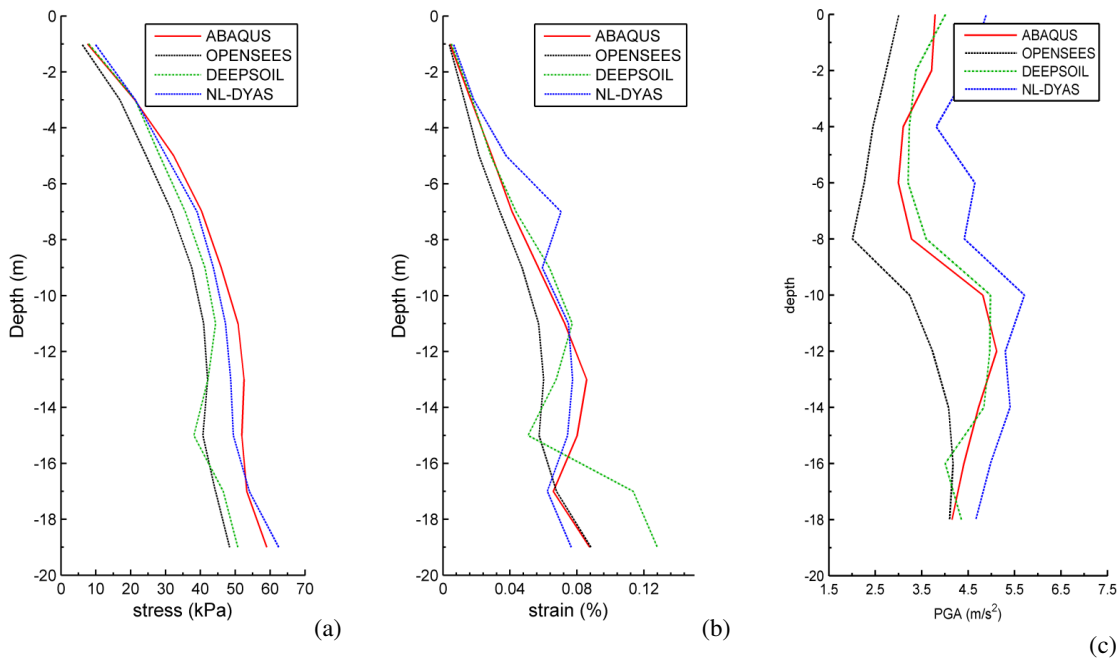


Figure 11. (a) Maximum stress (b) maximum stain (%) and (c) peak horizontal acceleration with depth for the non-linear soil response of profile P1

CONCLUSIONS

The paper studied the elastic, visco-elastic and non-linear response of two idealized soil profiles: a homogeneous single layer soil deposit of 20 m depth with constant shear wave velocity and a soil profile of 100 m depth with gradient distribution of shear wave velocity with depth. The profiles were subjected to a number of idealized (Ricker type) and really recorded time-history accelerations. The numerical simulations were performed with NL-DYAS, DEEPSOIL, OPENSEES and ABAQUS, incorporating several constitutive models. Results are presented in terms of transfer functions, peak ground accelerations, shear stresses and shear strains.

For both purely elastic and visco-elastic analyses all four codes predict quite similar responses in terms of peak acceleration amplitudes and frequencies even for low damping value. Some numerical problems were detected only for the case of low damping value and rigid bedrock. In frequency domain and especially in the frequency of interest in engineering practice (f<6.5Hz) all codes give similar results. In higher frequencies ABAQUS, OPENSEES and DEEPSOIL slightly underestimate the soil's resonant frequency or the amplitude value.

The comparison for the non-linear ground response analysis is not always equally satisfactory; the reason should be attributed to the differences in constitutive model and damping modeling between the codes. The most important conclusions of the present work are summarized as follows: (i) Bedrock simulation (e.g. rigid or elastic half space) have a negligible effect on the frequency characteristics of the computed time histories at soil surface; on the contrary they affect the peak ground acceleration amplitudes, with the rigid rock presenting clearly higher values. (ii) Comparisons of predicted response indicate that (a) in terms of frequency content ABAQUS, OPENSEES and DEEPSOIL result

in quite similar results (similar phase); (b) in terms of peak ground accelerations ABAQUS and DEEPSOIL predict similar values while OPENSEES and NL-DYAS give the lowest and highest values in almost all depths; (c) all four codes predict quite similar maximum shear stresses and strains values.The use of different numerical codes to estimate site effects may lead to considerable differences, which depend on several parameters not always related to the constitutive relationships used in each code to model the non-linear soil behavior. For example rigid bedrock should be avoided. The user should be aware of the limitations of each code and apply them consciously and according to the problem anticipated.

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