



Equivalent Linear and Nonlinear Site-Specific Ground Response Analysis of Pashto Cultural Museum Peshawar, Pakistan

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Abstract

Site-specific ground response analysis is needed for seismic hazard estimation, and its mitigation is required for seismically active regions of Pakistan. The work presented in the following includes response analysis of a specific site at Peshawar Khyber Pakhtunkhwa using equivalent linear (EL) and also nonlinear (NL) analyses. The site model was developed using the geotechnical investigation data from field investigation and subsequent laboratory tests on representative soil samples. The earthquake records compatible to the seismic activities in the target area are selected and matched to the target response spectra in order to obtain input excitation force for the base ground model. Several issues that are related to site-specific ground response analysis, i.e., shear strain, mobilized shear strength and ground acceleration along depth of soil model, are studied and compared for both EL and NL analyses. The analysis results are also further studied and compared for ground response spectra and amplification factor using both EL and NL analyses.

Keywords Site-specific ground response analysis · Equivalent linear (EL) analysis · Nonlinear (NL) analysis · Shear strain · Ground acceleration · Response spectra · Amplification factor (AF)

1 Introduction

Pakistan is in the geological expression of the Eurasian and Indian Plates. The Indian Plate is forced to sink under Eurasian Plate at a rate of 42 mm/year (Jain et al. 2000). The country experienced several disastrous earthquakes including 1945 Makran ($M_w=8.0$), 1931 Mach ($M_w=7.3$), 1935 Quetta ($M_w=7.4$) and the recent 2005 Muzaffarabad earthquake ($M_w=7.6$). This recent earthquake whose epicenter was about 19 km in northeast of Muzaffarabad according to Zaré and Paridari (2008) resulted in severe loss of life

and property in Eastern belt of Afghanistan, Khyber Pakhtunkhwa region of Pakistan, Azad Kashmir and Jammu and Kashmir region of India. According to official reports, more than 80,000 persons were killed in this devastating earthquake (Zaré and Paridari 2008). The existence of seismically active faults in the northern and southern areas will continue to have seismic hazard. Therefore, the assessed earthquake hazard and the associated risks along with its mitigation are of great importance for the sustainable development of the seismically active country of Pakistan.

Peshawar is recognized as the capital city of Khyber Pakhtunkhwa region of Pakistan and serves as the economic and urban activities center for the federally administrated tribal area. The city is located in the western Himalayan region. According to Building Code of Pakistan Seismic Provision, 2007 (BCP-SP 2007), Peshawar is in Zone-2B with PGA value of 0.16–0.24 g at rock surface. The peak ground acceleration (PGA) value at bedrock surface that is calculated for average shear wave velocity may not define a representative site-specific parameter. The ground response analysis should take into account the influence of soil medium on stress waves above the bedrock. The reason is that although the stress waves travel several kilometers through bedrock, in soils they travel up to a distance of

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100 m which indicates that the soil medium is an important factor in estimating the ground motion at foundation level. Evidence from recent earthquakes including the 1989 Loma Prieta, 1994 Northridge, 1995 Hyogoken-Nanbu and 1999 ChiChi indicates the importance of local soil conditions on the seismic activities in weathered formations at foundation level. The site-specific ground response analysis is important for the development of design response spectra that is further needed for the evaluation of liquefaction and seismic design of engineered slopes and externally stabilized soil systems.

One-dimensional ground response modeling is commonly practiced for the determination of site-specific seismic hazard response. The analyses are commonly carried out considering vertical SH wave propagation through horizontally layered soil from the bedrock using softwares such as SHAKE (Idriss and Sun 1992) and DEEPSOIL (Hashash 2012). There are several studies available worldwide such as Seed and Idriss (1970), Grasso and Maugeri (2005, 2009), Phillips and Hashash (2009), Bonaccorso et al.

(2005), Monaco et al. (2011), Akhila et al. (2012), Cavallo et al. (2012a, b, 2013), Maugeri et al. (2011) and Naik and Choudhury (2014) where site-specific soil properties have been taken into account. Analyses were conducted to determine site-specific response analysis for Kashiwazaki-Kariwa Nuclear Power Plant, Japan, by Shylamoni et al. (2014). The results were in close agreement with the after-shock results obtained from field study. In the context of Indian Plate, there are also several site-specific studies available (GovindaRaju et al. 2004; Bhatti et al. 2011; Thaker et al. 2009; Shukla and Choudhury 2012) for which the sites have been characterized based on the shear wave velocity V_s estimated from the empirical correlations of (SPT) number, N . Although several studies are available on seismic hazard analysis (Monaliza et al. 2004) in Pakistan, there is complete scarcity of the available literature on the topic of site-specific seismic hazard analysis and the only case study available is that of Mahmood et al. (2016) for the collapsed Margalla Tower in Islamabad. Hence, the present study is conducted

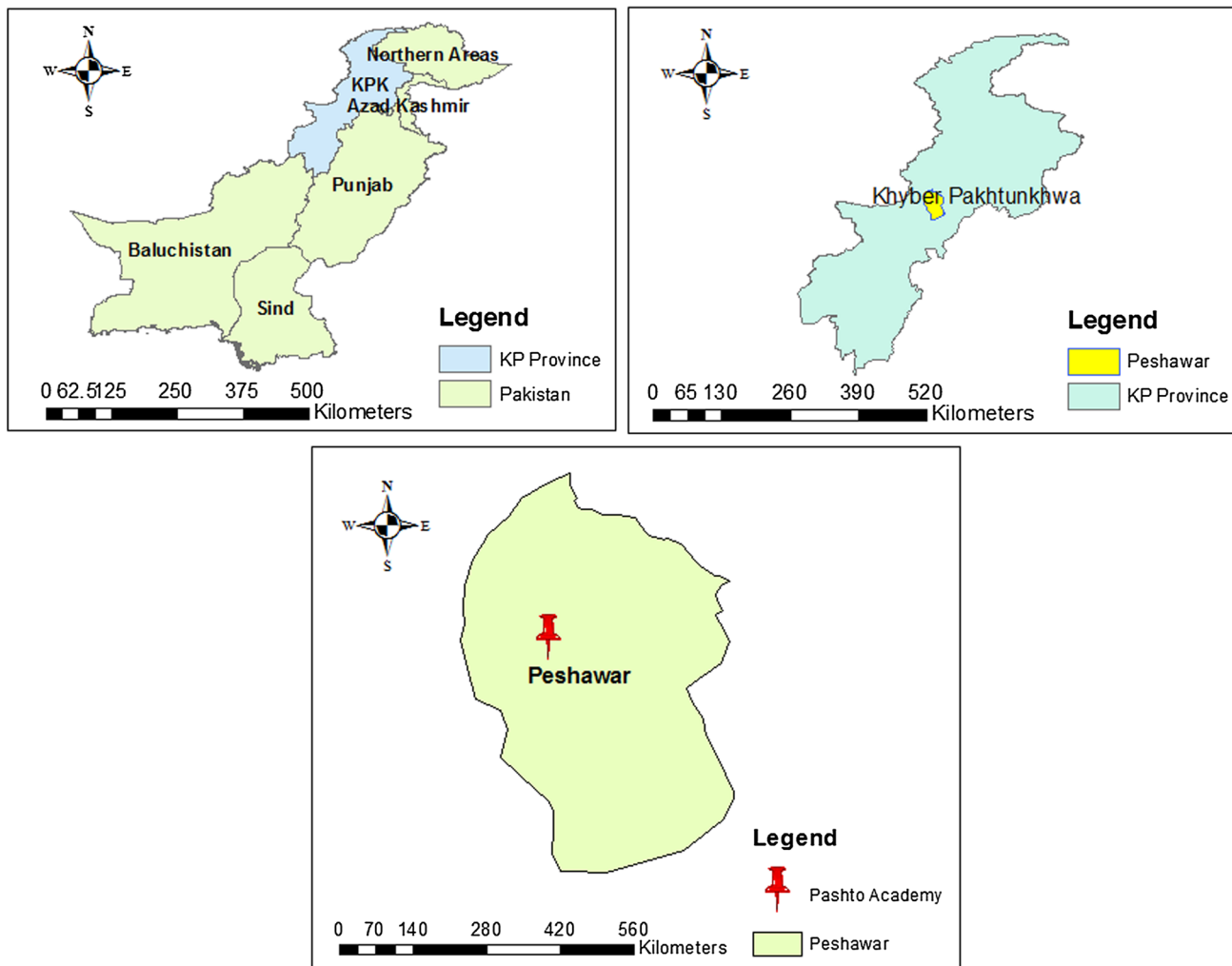


Fig. 1 Location map of target site

to evaluate ground response analysis for a selected location in Peshawar (Fig. 1) using geotechnical data obtained from a borehole. Compatible bedrock motions were derived from the Pacific Earthquake Engineering Research Center (PEER) NGA West 2 database for the Peshawar region (Ancheta et al. 2014).

Selection of appropriate analysis method based on linear, equivalent linear (EL) and nonlinear technique (NL) is considered one of the key steps in the determination of seismic hazard analysis. Several studies including Zalachoris and Rathje (2015) and Kaklamanos et al. (2015) showed that the NL method would be needed for comparison with EL method when the soil deposit is subjected to high strain rates in the range of 0.1 to 1%. According to Kaklamanos et al. (2015) for six sites at Kiban-Kyoshin, the predicted site response analysis was significantly improved for strain level above 0.1% when EL and NL models were used. Eskandarinejad et al. (2017) investigated several sites around Shiraz, Iran, with clay as dominant strata, to investigate the effect of the shear strength, input ground motion intensity and soil condition on the divergence between EL and NL spectral accelerations. They also compared shear strain index for less than 5 Hz frequencies with the results provided by Kim et al. (2016).

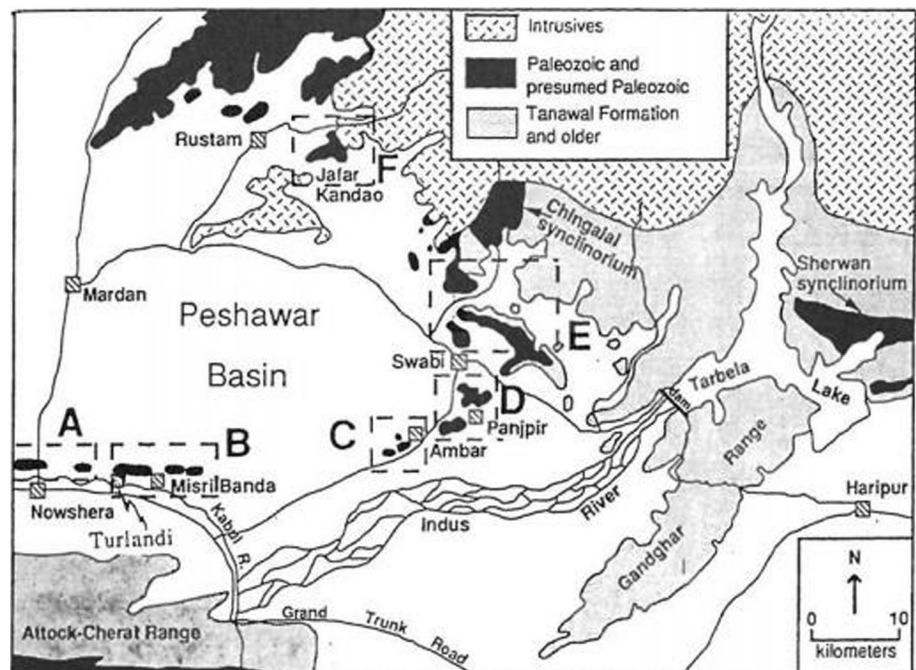
In the case of site response analysis, the surface amplification can be significantly affected by the impedance contrast and location of bedrock. Several researchers including (i.e., Eskandarinejad et al. 2018; Baise et al. 2016; Nath and Jakka 2012; Hashash et al. 2014 and others) have presented the effect of bedrock depth on ground response analysis. Nath and Jakka (2012) studied the effect of different parameters

such as impedance contrast of bedrock and surficial layer, depth of bedrock and shear wave velocity at five sites in Delhi, India. According to their results, the amplification of seismic waves is affected by depth of bedrock at different frequencies. Moreover, in the case of shallow thickness above bedrock the amplification of seismic waves occurs at higher frequencies, while for high depth the amplification occurs at lower frequencies. In the absence of site-specific stratigraphic profile, Baise et al. (2016) used a shape function (SF) and “kriging-with-a-trend” interpolation method to estimate depth of bedrock for Boston area. Their model was based on the stratigraphic information of over 500 boreholes paired that contained information of geology and geomorphology of the study area. By using this model, Baise et al. (2016) estimated that the mean short period and mean intermediate-period amplification used in design codes may underpredict soil amplification in strong impedance contrast environment such as Boston. In the present study, as there is a scarcity of geological and geotechnical information for the target site, with limitations the bedrock is set at the base of borehole.

2 Peshawar Basin Geological Setting

Peshawar basin is located at the salt range to the north of Himalayan deformation and south of Main Mantle Thrust between the Indian Plate and the Kohistan island arc terrane (Hussain et al. 1991). The Peshawar basin (Fig. 2) tectonic setting has transition from a sedimentary fold-and-thrust belt in the south to metamorphic terrane in the north (Hussain

Fig. 2 Geological map of Peshawar basin showing areas of different stratigraphic columns (after Hussain et al. 1991)



et al. 1991). Peshawar basin was formed when about 300 m sediments were deposited in the Pleistocene era. This deposition of sediment was due to the ponding drainage because of rising of the Attock–Cherat Range. The unlithified sediments are mostly lacustrine silts with gravel and fluvial sand.

According to Aslam et al. (2006), Peshawar valley is covered with unconsolidated deposits of sand, silt and gravel. Ahmad et al. (2013) have conducted a profiling and zoning study for Peshawar basin using geographical information study and have divided the basin into different zones based on the geotechnical data. Their results show the presence of gravel, silts and clay at different locations in shallow depth.

3 Shear Wave Velocity and Subsurface Profile

It is common practice to obtain the shear wave velocity from the results of standard penetration test (SPT-N value). Among researchers, Seed and Idriss (1981), Imai and Tonouchi (1982), Jafari et al. (1997), Kiku et al. (2001), Hasancebi and Ulusay (2006) and Dikmen (2009) have developed shear wave velocity and SPT-N value relationships for different kinds of soils. The variation of shear wave velocity with depth based on the relationships developed by these researchers along with average shear wave velocity that is used in one-dimensional ground response analysis is shown in Fig. 3. The site is stiff S_D according to BCP-SP (2007).

The parameters needed for ground response analysis were obtained from a 10-m-deep borehole at a selected location

Fig. 3 Shear wave velocity profile along depth of borehole

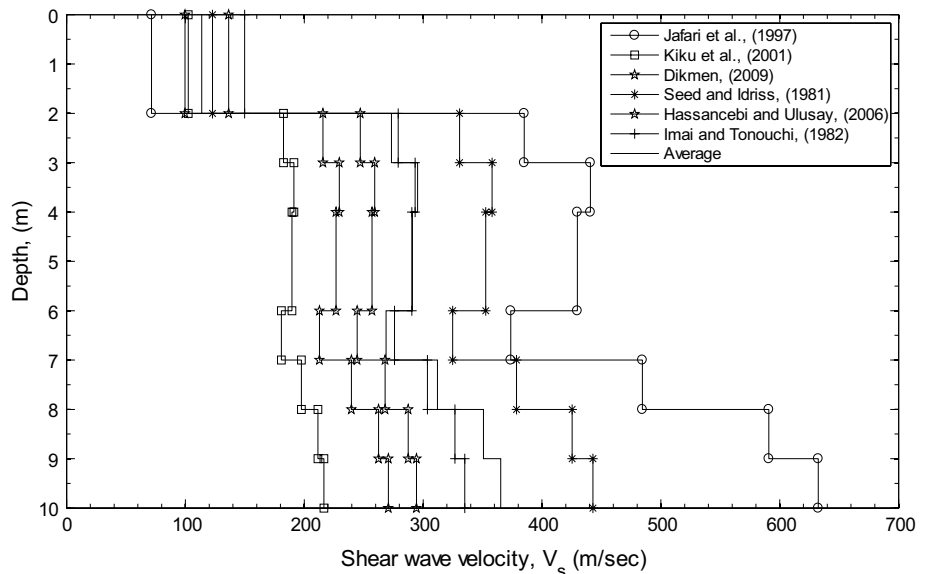


Table 1 Subsoil profile parameters used in one-dimensional ground response analysis

Layer depth (m)	SPT-N, values	Plasticity index, PI (%)	V_s (m/s)	Density (kN/m ³)	Average shear wave velocity (m/s)	Soil as per BCP (2007)
2	4	5	188	16	$v_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} = 269$	Stiff soil, S_D (average shear wave velocity 175 to 350 m/s)
1	29	9.8	273	17.5		
1	34		295			
1	33		291			
2	33		291			
1	28		268			
1	38	9.8	311	17.7		
1	48		350			
1	52		365			

of Pashto Cultural and Heritage Museum at University of Peshawar, Pakistan (Table 1). The bedrock depth at Peshawar basin is variable, and for simplicity, it is set up at the depth of borehole assuming unit weight of 22 kN/m^3 with a shear wave velocity of 760 m/s and 2% damping. The shear modulus degradation and damping curves are based on the plasticity index (PI) proposed by Vucetic and Dobry (1991) for 5% damping value.

4 Seismic Hazard and Input Ground Motions

Input ground motion is considered very important in the design of civil engineering structures for dynamic analysis. It is the main unknown of ground response analysis. The ideal conditions for a complete site response analysis are to model the fault rupture mechanism (Wells and Copper-smith 1994) at the surface, the propagation of stress waves from the fault rupture to a particular site from bedrock (Kramer 1996) and distance from the source (Boore and Atkinson 2008). The Worldwide Standard Seismograph Network (WWSSN) in 1961, Global Digital Seismometer Network (GDSN) and Global Seismographic Network (GSN) in 1980 have dramatically improved the understanding of earthquake and tectonic process. Regional arrays of seismographs are now available in most seismically active countries (Kramer 1996). Pakistan lacks regional array of seismographs to record and analyze strong motion records of earthquake. The worldwide recorded accelerograms are compatible with the earthquake characteristics of Pakistan and are thus a good alternative for ground response analysis.

The deterministic seismic hazard study of Waseem et al. (2013) identified several faults in the 100 km radius of Peshawar that can produce earthquake potential of

magnitude 7.1 and above. According to their study, most of these seismogenic faults have reversed faulting mechanism. In the present study, the earthquake records compatible to the seismic hazard of Peshawar were selected from PEER strong motion database. A suite of 11 earthquake records were selected that fit the target response spectrum. The target response spectrum for Peshawar is based on BCP-SP (2007) for zone 2B that has a zoning factor of 0.2 . The earthquake characteristics along with scaling factor to target spectrum are given in Table 2. Furthermore, Fig. 4 shows the response spectra of a suite of input motions along with target response spectrum.

A suite of input accelerograms were then further matched to target response spectrum and used as base input motions in the site response analysis. Figures 5 and 6 show,

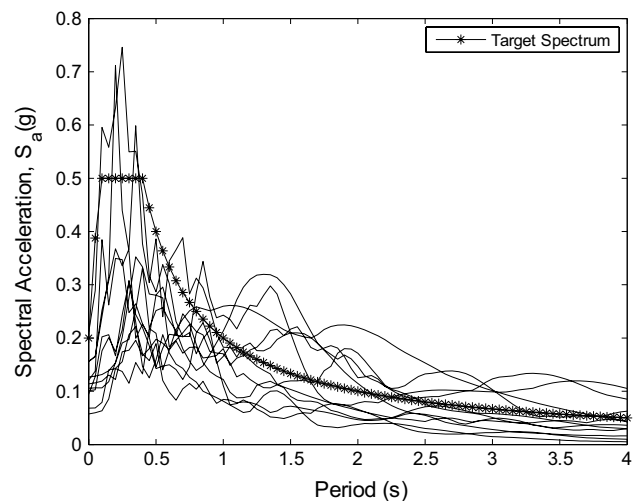


Fig. 4 Response spectra for a suite of input motions fitted with target response spectrum

Table 2 Characteristics of earthquake records from PEER strong motion database

Earthquake name	Station name	Faulting mechanism	Maximum potential magnitude (M_w)	Joyner and Boore distance, km (R_{JB})	Scaling factor
Kern County	Taft Lincoln School	Reversed faulting	7.36	38.42	0.8416
San Fernando	LA—Hollywood Stor FF		6.61	22.77	0.6721
San Fernando	Palmdale Fire Station		6.61	24.16	1.1052
San Fernando	Pasadena—CIT Athenaeum		6.61	25.47	1.2627
San Fernando	Santa Felita Dam (Outlet)		6.61	24.69	1.4116
San Fernando	Whittier Narrows Dam		6.61	39.45	1.4778
Point Mugu	Port Hueneme		5.65	15.48	1.6607
Friuli_Italy-01	Codroipo		6.5	33.32	1.6235
Friuli_Italy-01	Buia		5.91	10.99	1.3856
Friuli_Italy-01	San Rocco		5.91	14.37	1.8582
Santa Barbara	Santa Barbara Courthouse		5.92	0	1.1897

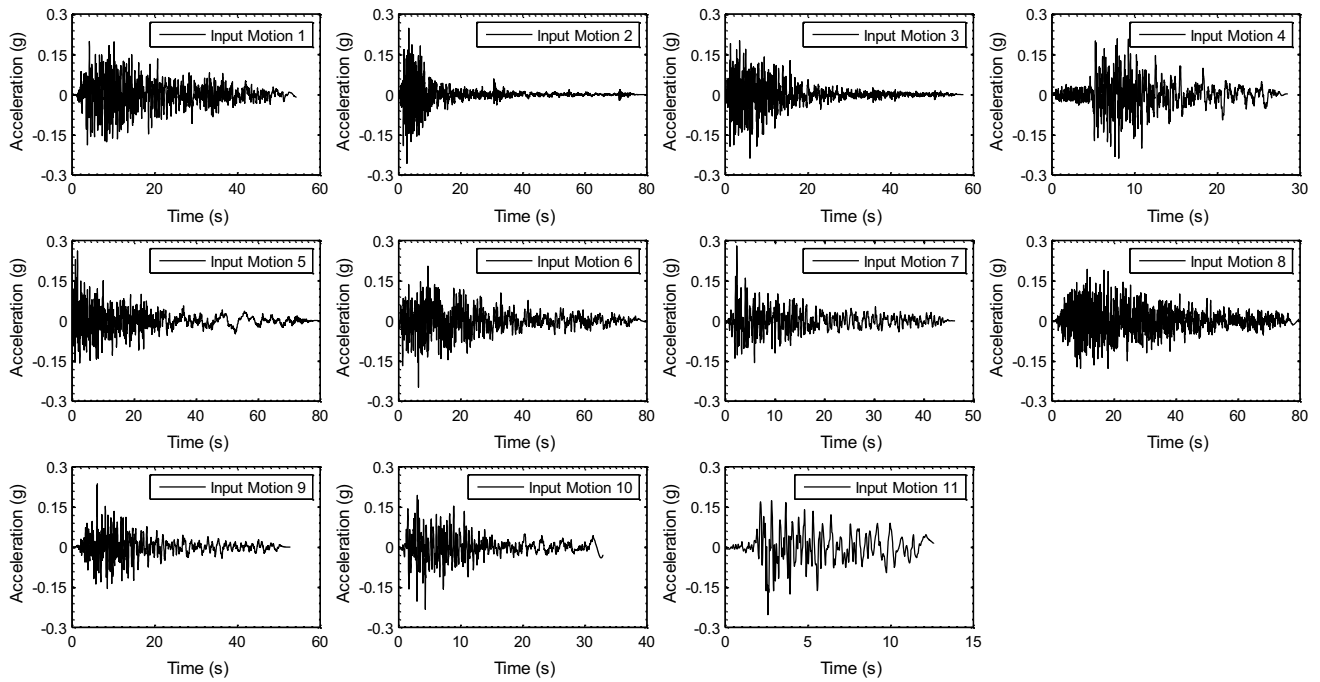


Fig. 5 A suite of input accelerograms used in ground response analysis

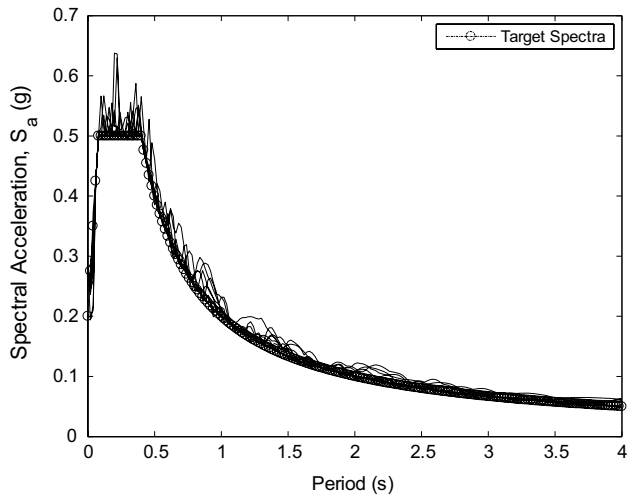


Fig. 6 A suite of input motions of response spectra matched with target spectra

Table 3 Parameters for a suite of input ground motions

Earthquake name	Input motion	PGA (g)	v_{max}/a_{max} (s)	Arias intensity (m/s)
Kern County	1	0.20	0.11	1.54
San Fernando	2	0.26	0.10	0.82
San Fernando	3	0.24	0.07	1.06
San Fernando	4	0.24	0.08	1.00
San Fernando	5	0.26	0.19	1.29
San Fernando	6	0.25	0.30	1.71
Point Mugu	7	0.28	0.10	0.85
Friuli_Italy-01	8	0.19	0.29	2.06
Friuli_Italy-01	9	0.23	0.09	0.85
Friuli_Italy-01	10	0.23	0.08	0.82
Santa Barbara	11	0.25	0.10	0.56

respectively, the accelerograms and response spectra of a suite of matched input motions used in the site response analysis. The intensity and frequency parameters for a suite of input motions are given in Table 3.

The PGA is commonly used in the design of ground response spectra for a particular site. The parameter has a natural relationship to inertial force. According to Table 3, input motion 7 has the highest, while input motion 8 has the lowest PGA value. The quantity $2\pi v_{max}/a_{max}$ can be interpreted

as the time period of an equivalent harmonic wave. Table 3 shows that input motion 8 has the longest time period (lowest frequency) among a suite of input motions. Arias intensity is another parameter of input motions and is the measure of the intensity of acceleration integrated as a function of time. According to Table 3, input motions 8 and 11, respectively, have the highest and lowest arias intensity value among a suite of input motions.

5 One-Dimensional Ground Response Analysis

There are theoretical reasons that ground surface motion is influenced by local soil conditions. In most geotechnical sites, the soil density and shear wave velocity are smaller at the surface than those in greater depths. If the material damping is neglected, then there must be constant energy flux $\rho v_s u^2$ along the depth to ground surface. As the density ρ and shear wave velocity v_s decrease toward the surface, this means that the soil particle velocity, u , increases toward the surface. The local site condition can thus affect the PGA and spectral acceleration value and can be different from that of the bedrock value.

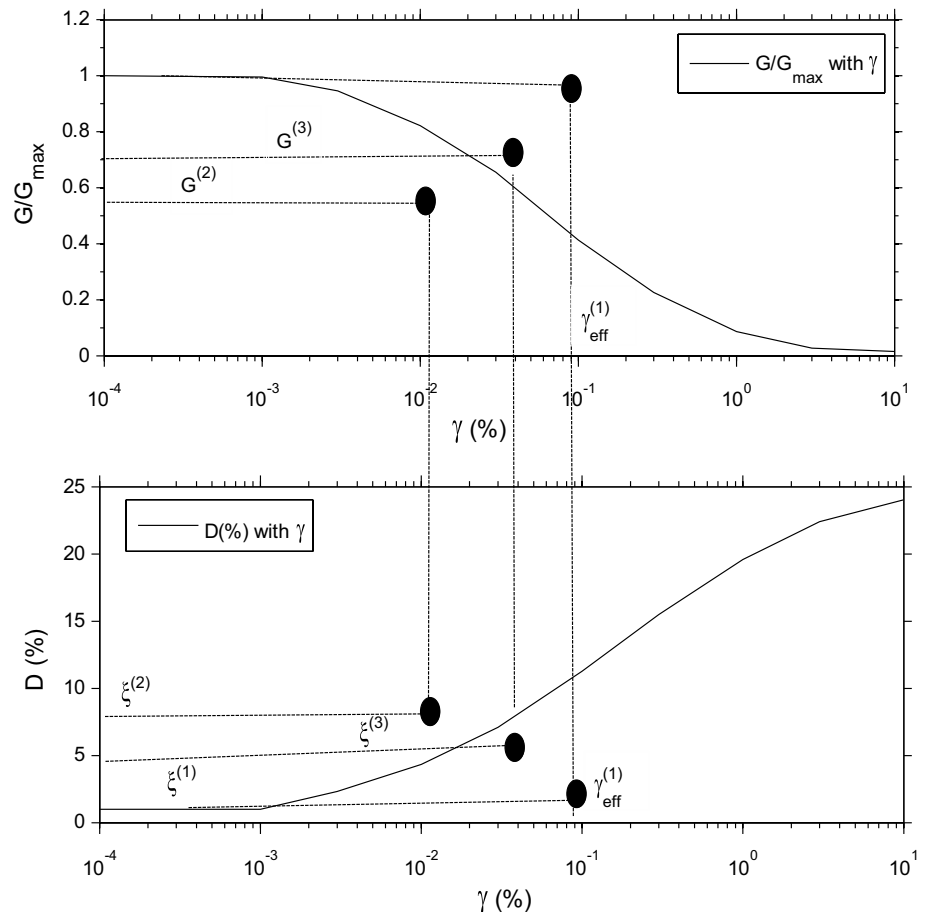
One-dimensional ground response analysis is used to solve the problem of propagation of seismic stress waves through layered soil deposit. The soil deposit is approximated through a Kelvin–Voigt model for constant elastic shear moduli and viscous damping in DEEPSOIL software (Hashash 2012). The software features both the NL and EL ground response analyses.

The equivalent linear is used for which the spring and dashpot parameters are calculated based on the secant shear

modulus and the damping ratio for a given level of shear strain using equivalent linear iterative procedure (Kramer 1996). The iteration procedure in EL analysis is shown in Fig. 7. Figure 7 shows that the initial value of shear modulus $G^{(1)}$ and damping ratio $\xi^{(1)}$ predicts a reference shear strain $\gamma_{eff}^{(1)}$. The strain is, however, greater at this value of shear modulus $G^{(1)}$ and damping ratio $\xi^{(1)}$, and an iteration is thus needed. The next iteration is then made for shear modulus $G^{(2)}$ and damping ratio $\xi^{(2)}$ compatible with reference shear strain ratio $\xi^{(2)}$. The process is repeated until the strain compatible values of G and ξ are obtained.

In the case of NL model, the soil stratum is discretized into several layers using multi-degree-of-freedom lumped parameter model. In the lumped parameter system, the individual layer is represented by a mass, nonlinear spring and dashpot for viscous damping. The soil nonlinear properties are determined by direct numerical integration method in time domain. At each time step of loading, the stress–strain relationship is evaluated to obtain the appropriate soil properties at that time step. The small incremental time steps are thus used to get the complete nonlinear soil properties at the end of loading. In the ground response analysis, the hyperbolic model is used to define the backbone curve in order to find the modulus reduction G/G_{max} as follows:

Fig. 7 Iteration procedure in EL analysis



$$\frac{G}{G_{\max}} = \frac{1}{1 + \beta \left(\frac{\gamma}{\gamma_r} \right)^a} \quad (1)$$

where $\alpha = 0.92$ and $\beta = 1$ and γ_r is the pseudo-reference strain and describes the backbone curve at small strain.

The present study uses the pressure-dependent hyperbolic model for EL and NL site response analyses. This model was developed by Matasovic (1993) and is based on the hyperbolic model proposed by Kondner (1963). At small strains, the model results in zero hysteresis damping. These small strains damping are added even at small strains to simulate nonlinear soil behavior model. The soil degradation and damping curves used are those proposed by Vucetic and Dobry (1991) for 5% damping value. Furthermore, the procedure of best fit is used to find the parameters for these curves.

The evaluation of subsurface profile, soil type and properties is needed for a complete geotechnical site characterization (Hashash 2012). The parameters as given in Table 1 are used for the site response analysis.

6 Analysis Results and Discussion

The site fundamental period, T_n , of a homogeneous soil deposit with height, H , and constant shear wave velocity V_s is written as:

$$T_n = \frac{4H}{V_s} \quad (2)$$

The site fundamental period for stratified soil deposit with n number of layers with different shear wave velocities v_{si} and heights h_i can be written as:

$$T_n = \frac{4H}{\left[\frac{1}{H} \left(\sum_{i=1}^n \frac{h_i}{V_{si}} \right) \right]} \quad (3)$$

The natural fundamental frequency of the site is calculated from the reciprocal of fundamental natural period (Eq. 2). The fundamental natural period is 0.15, and fundamental frequency of the site obtained is 6.74 Hz, respectively.

In the case of EL analysis, the initial shear strain time history is calculated based on the initial value of shear modulus and damping of soil layer. These soil properties are set constant from the start to the end of input motion. A representative shear strain of 65% is then selected and used to find the degraded shear modulus and damping. These soil properties are then used in the next iteration process until the values converge to the target degradation curves. According to Lasley et al. (2014), the shear strain in any layer γ_i with depth z_i for angular frequency ω can be calculated as follows:

$$\gamma_j(z_i, \omega) = ik_i^* \{ A_i(\omega) \exp(ik_i^* z_i) - B_i(\omega) \exp(-ik_i^* z_i) \} \quad (4)$$

where k_i^* is defined as the complex shear wave number and is equal to $\omega \sqrt{\rho/G^*}$, while $A_i(\omega)$ and $B_i(\omega)$ are the amplitude of the input motion waves in the upward and downward directions in soil layer. The frequency-independent complex shear modulus for a soil layer is dependent on damping value ξ and is equal to $G(1 + i2\xi)$.

In the case of NL analysis, initially the soil deposit is at rest; thus, the particle velocity and shear stress for all layers are zero. As the input motion travels through the rock and reaches the soil–rock interface at incremental time Δt , a nonzero value for particle velocity and thus particle displacement are obtained. At the subsequent time steps, the particle displacement in each layer will also take on a nonzero value. At each time step, the incremental displacement is thus calculated and summed to find the total displacement in each layer. The resulted shear strain in a sub-layer i is then given by:

$$\gamma_{i,t} = \frac{u_{i+1} - u_{i,t}}{\Delta z} \quad (5)$$

where $u_{i,t}$ and u_{i+1} is the total displacement at the top of layer i and $i + 1$ with thickness Δz .

The input motion applied at bedrock level induces shear stresses at different layers during its propagation to the surface that causes shear strain and thus degradation in these soil layers. From the previous discussion, it is thus clear that during input motion waves through soil layers the EL and NL analyses use different procedures to calculate the soil dynamic properties. Thus, the developed shear strain along the depth of soil profile for a suite of input motions may also be different for both EL and NL analyses. Figure 8 shows the developed shear strain along the depth of soil profile for a suite of input motions. Figure 8 shows that the shear strain in the case of NL analysis is slightly greater than that calculated from EL analysis.

The mobilized shear strength at each point of soil is dependent on the shear strain γ and the corresponding shear modulus. The shear strain values that are calculated for different layers as already discussed for both EL and NL analyses are thus used to calculate the mobilized shear strength as follows:

$$\tau = \rho V_s^2 \frac{G}{G_o} \gamma \quad (6)$$

The mobilized shear strength for a suite of input motions in the case of both EL and NL analyses is shown in Fig. 9. According to Fig. 9, the mobilized shear strength is decreasing toward surface. Furthermore, there is also discrepancy in mobilized shear strength among a suite of input motions and the analysis method used. The reason for this can be that

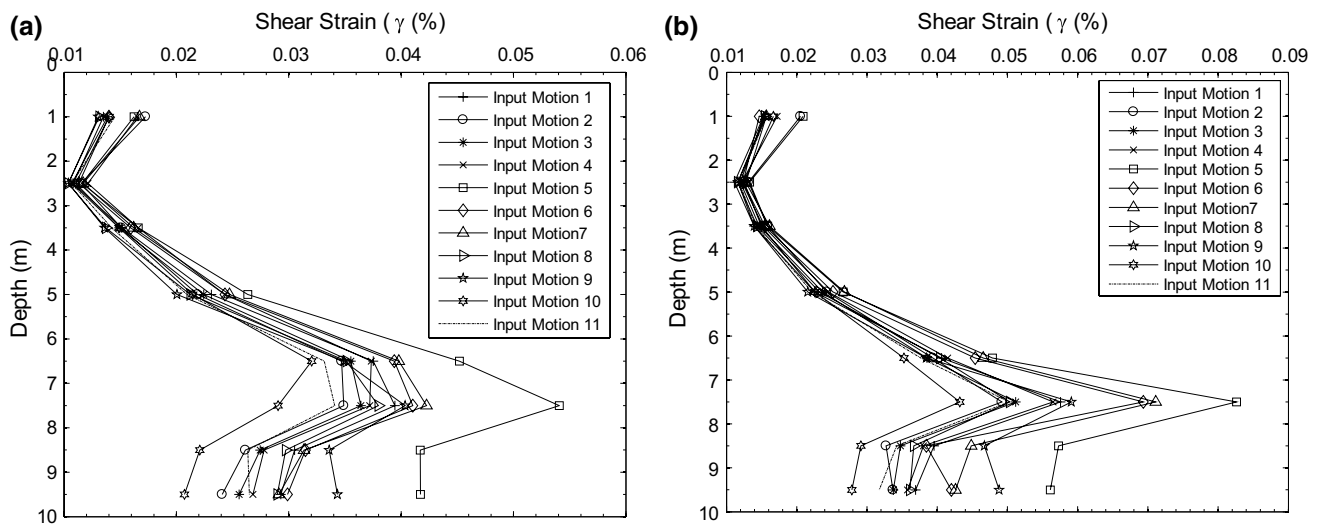


Fig. 8 Shear strain developed along depth of profile, for a EL and b NL analyses

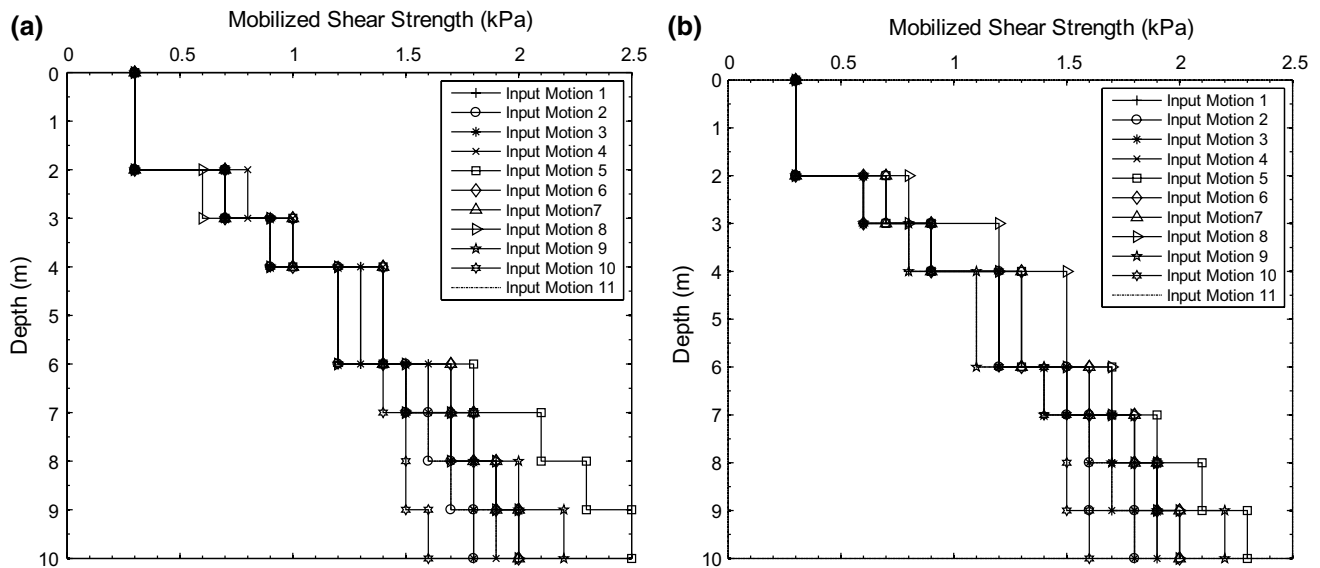


Fig. 9 Mobilized shear strength (kPa) along depth of profile for a EL and b NL analyses

for a specific site that has a constant ρV_s^2 , for each layer the discrepancy in results can be attributed to the different shear strain developed and the corresponding shear modulus used by a suite of input motions.

The response spectrum is commonly used in the design of earthquake-resistant structures. This frequency content of the motion is reflected as spectral acceleration at surface and is often used as one of the important parameters to estimate the ground response. The response spectrum for a single-degree-of-freedom (SDOF) system can be defined as the maximum response when it is subjected to seismic

loading. This response is a function of natural period (or natural frequency) and damping ratio of the SDOF system. Figure 10 shows the surface response spectra in comparison with bedrock spectra for a suite of input motions for both EL and NL site response analyses. The median surface response spectra show that almost all suite of input ground motions are amplified near the site fundamental period (0.15 s).

The amplification factor is defined as $(AF = S_{a,surface}/S_{a,bedrock})$ and is also equal to the ratio of the amplitude of surface and bedrock. During the propagation of seismic waves, a standing wave is developed at each natural frequency of the soil deposit that can cause amplification at

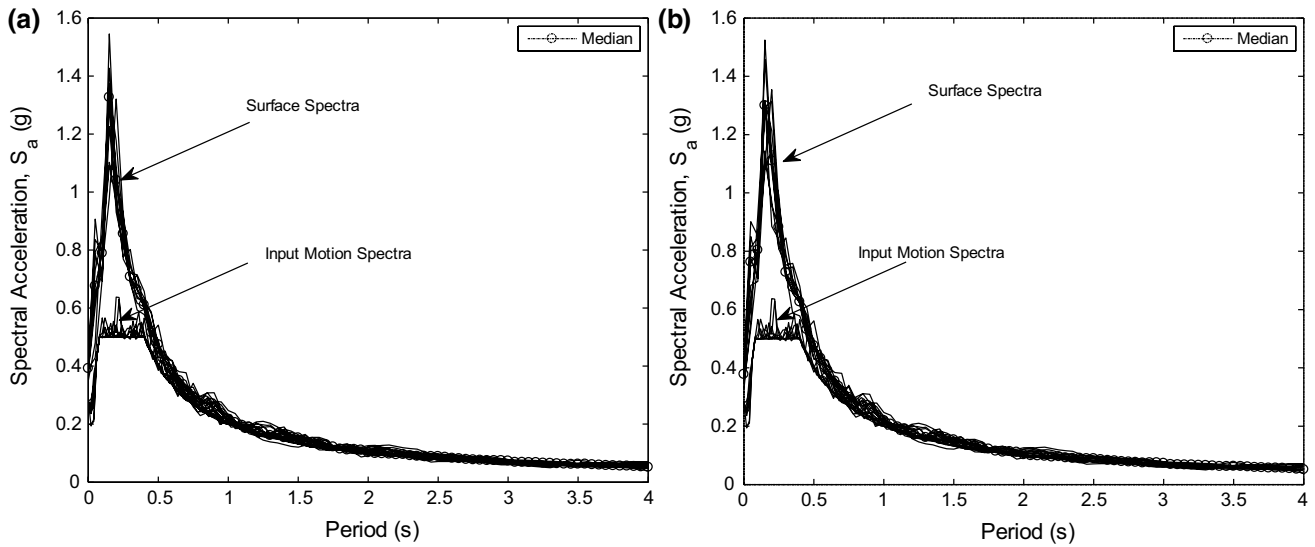


Fig. 10 Compared response spectra for a suite of input and surface motions, **a** EL and **b** NL analyses

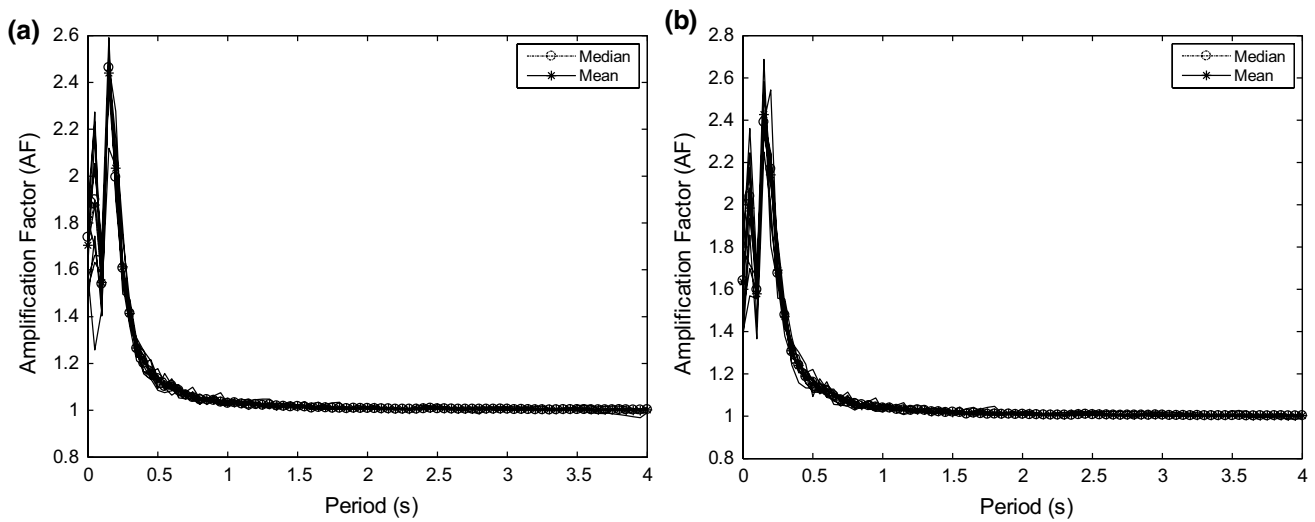


Fig. 11 Comparison of amplification factor (AF) for **a** EL and **b** NL analyses

the surface. Above the fundamental frequency, some part of the soil deposit is moving in one direction and the remaining is moving in the opposite direction (Kramer 1996). This phenomenon is important to evaluate inertial forces in the soil mass required for seismic stability analysis. Figure 11 shows the amplification factor for a suite of input motions in the case of both EL and NL response analyses. According to Fig. 11, the median and mean amplification factor is about 2.5 that occurs near the fundamental period (0.15 s) of the site.

The amplification of the rock motion by the soil deposit at fundamental period of the site may subject structures to

large dynamic forces and thus be damaged. Furthermore, if the fundamental period of the site also coincides to that of the structure, a double-resonance condition may be developed that causes devastating damage.

The transfer function in ground response analysis is used to relate the different parameters such as acceleration, velocity and displacement at different layers of a soil deposit to the input motion such as bedrock acceleration. The transfer function shows how different parameters in a layered soil deposit are amplified or de-amplified as a function of frequency. In the present case for layered deposit, this transfer function can be defined as:

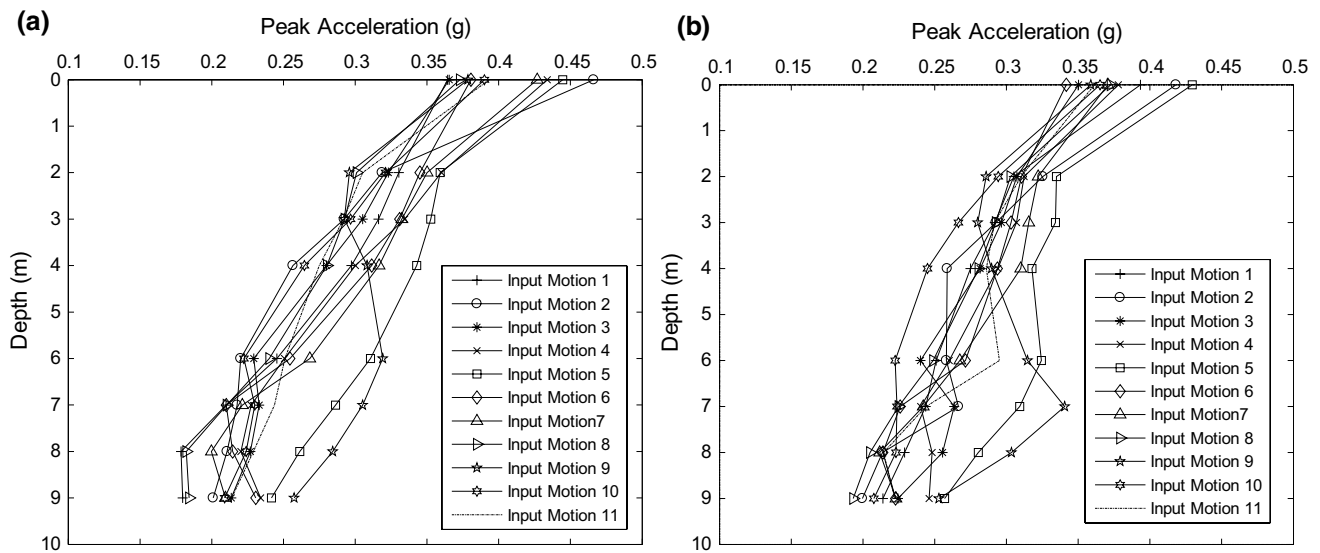


Fig. 12 Variation of PGA value along depth of borehole, **a** EL and **b** NL analyses

$$F = \frac{1}{\cos(\omega H/v_s^*) + i\alpha_z^* \sin(\omega H/v_s^*)} \tag{7}$$

where $v_s^* = v_s(1 + i\xi)$ is the damped frequency with soil height H and $\alpha_z^* = \rho v_s^*/\rho v_r^*$ is the complex impedance ratio.

The transfer function is used to find the motion in one layer from the motion of any other layer.

The maximum amplitude of a particular ground motion (i.e., PGA) has natural relationships with inertial force and is commonly used to describe ground motions. Based on the transfer function, the variation of PGA along the depth of borehole can thus be computed as shown in Fig. 12. Figure 12 shows that high surface PGA value in comparison with the bedrock is obtained for all the suite of input motions. Figure 12 shows that in the case of EL slightly higher PGA values along the soil profile are obtained in comparison with NL analysis. The reason may be that both EL and NL analyses use different approaches to estimate the degradation curves and shear strain.

7 Conclusions

The EL and NL one-dimensional ground response analyses were conducted for a selected location at Peshawar Pakistan. The following are the conclusions drawn from this research:

- The site average shear wave velocity is 269 m/s that according to BCP-SP (2007) is classified as site S_D .
- The EL and NL analysis methods use different approaches to calculate the shear degradation and shear

strain at the soil deposit. This different approach of methods thus resulted in slightly larger shear strain for NL in comparison with EL method. The induced shear strength is dependent on shear degradation, and thus, the corresponding strain also resulted in slightly different results along the depth of soil deposit.

- The response spectra at surface showed that all the suite of input motions were amplified near the site fundamental period (0.15 s).
- The same mean and median amplification factor (AF) of 2.4 was obtained at the site fundamental period (0.15 s) for both EL and NL analyses.
- The transfer function can be used to correlate the PGA values at different layers of soil deposit. The analysis results showed that the EL analysis resulted in slightly higher PGA values as compared to NL analysis.

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