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Full Length Research Paper

Nonlinear site response evaluation procedure under the strong motion: A case study of Miyaneh, Azarbayjan Sharghi Province, Iran

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The objective of a site response analysis is to estimate the ground shaking intensity during an earthquake. The Miyaneh city and its suburban areas are located in the northwest of Iran in Azarbayjan-Sharghi province. This area is prone to high seismic risk due to the presence of several active faults. In this paper, a case study on ground response analysis of a geotechnical site in Miyaneh region during the Ardabil earthquake (28 February, 1997, M_w6.1) is presented. For site characterization, deep site investigations have been undertaken and a seismic geotechnical procedure for the proposed bridge over the rivers at the mentioned site, which is performed for Iran railway network, subjected to earthquake provokes, has been notified. The effect of nonlinearity on site response analysis for the selected site with the assumption of elastic and rigid half space bedrock (by use of nonlinear standard hyperbolic model) was evaluated and the results obtained over a number of geotechnical areas were compared to each other. Test of the capability of designed computer codes by authors, namely; "Abbas converter", which describes and evaluates the nonlinearity of the subsurface soil conditions encountered at the sites to analyze, evaluate the obtained test, site response and quantify the site effect on the surface over a number of geotechnical areas, were the targets of this study. The results clearly showed that the effect of bedrock and local soil conditions on soil behavior under the earthquake excitation is one of the main effective factors on computed response spectra in ground response prediction. The key factor in this work was to develop and use "Abbas converter". It has several advantages, which can make and render the study easier, than previous studies done, and take over the encountered problem.

Key words: Miyaneh city, "Abbas Converter", Ardabil earthquake, site response, site amplification.

INTRODUCTION

Iran is located in the south west of Asia in the Middle East and has a special strategic location. During the past centuries, it has operated as a connecter between the gap of west and east by having the good pass; and since transportation and communication are considered as a prerequisite in any economic development, Iran's authorities have paid a special attention to the development of transportation and communication, especially via railways. The Tabriz-Jolfa line (146 km) in 1914, the Sufiyan-Sharafkhane (53 km) in 1916 and the Mirjaveh-Zahedan (93 km) in 1920 were built in Iran. In 1939, the Iranian Railway network with 1392 km long, as a connection link between Bandar Torkman on the Caspian Sea and Bandar Imam Khomeiny (Bandar Shahpur) on the Persian Gulf was opened, which became one of the supply routes for war material during the Second World War. This network traverses many mountain ranges and is full of spirals. Much of the terrain was unmapped when construction took place and its geology was unknown. Several stretches of line, including tunnels, were built through unsuitable geology,

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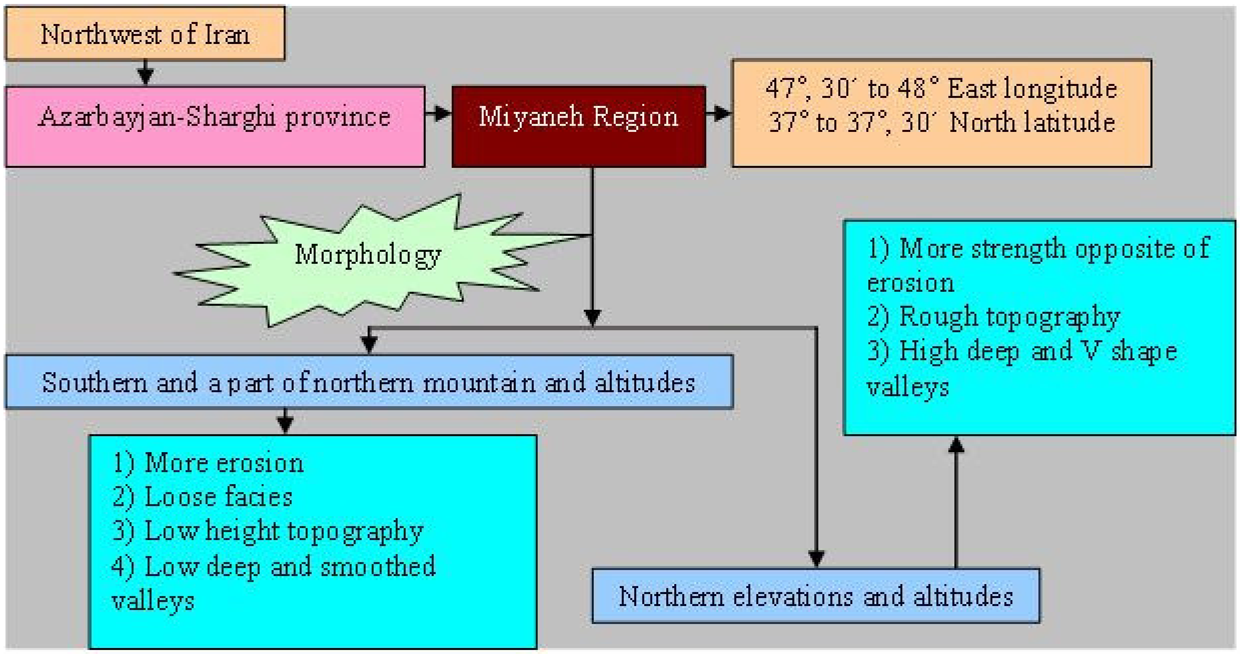

Figure 1. Location and morphological characteristics of the selected area.

and had to be replaced even before the line opened. Nevertheless, the line was completed ahead of schedule. In 2008, the IR operated 11106 Km of rail with a further 18900 Km in various stages of development. The Jolfa–Tabriz line is electrified (148 km) and up to 2006, the most majority of the engines were diesel-powered. The government plans to transport 3.5% of the passenger volume and 8.5% of the freight volume by rail. The railway network expands by about 500 km per year according to the Ministry of R and T.

As shown in Figure 1, the Miyaneh region with 47˚, 30´ to 48˚ East longitude and 37˚ to 37˚, 30´ North latitude is located on the Northwest of Iran in Azarbayjan-Sharghi province. On the basis of geomorphological point of view, the mentioned area is divided into two portions. The southern and a part of northern mountain and altitudes with marl, sandstone and conglomerate deposition has more erosion because of loose facies and has low height topography with low deep and smoothed valleys. The northern elevations consist of volcanic lavas, which because of stiffness and hardness of constituent materials with more strength opposite of erosion have a rough topography with high deep and V shape valleys. Almost all the region outcrops are Cenozoic volcanic and sedimentary rocks and are found only in a small portion of Ghafelankuh Mountains due to faulting. As a result, Permian-Terries carbonate sediments have exposures. This area is an active seismic belt which is located in Alborz-Azarbayjan seismotectonic province.

By ignoring the historical earthquakes and referring to the National Geodatabase of Iran website, the happened earthquakes with magnitude more than 4.5 are given in Table 1. The Ardabil earthquake (M_w 6.1, depth of 10 Km and 38.075 N, 48.050 E epicentral coordinates) was a destructive earthquake that occurred on 28 February, 1997 at 12:57 UTC (4:27 p.m. Iran Standard Time) and lasted for 15 seconds (Reuters (cable News Network), 1997-03-01, Reuters (cable News Network). 1997-03-04). Its epicenter was located near the city of Ardabil in the northwest of Iran and at least 1,100 people were killed; 2,600 injured; 36,000 homeless; 12,000 houses damaged or destroyed and 160,000 livestock killed in this area of Iran. Severe damage was observed to roads, electrical power lines, communications and water distribution systems around Ardabil (Person, Waverley L. (2008-07-16). Hospitals and other medical buildings were overflowing with patients as a result of the earthquake. More than 83 villages experienced some form of damage (Reuters (cable News Network), 1997-03-01). Within the village of Villadareh, 85 corpses were recovered from the rubble. In Varania, another small village near the epicenter that had previously had a population of 85, all but 20 residents had perished (Reuters (cable News Network), 1997-03-01).

In this study, by using geological, geophysical and geotechnical data with softwares combination such as Seismosignal, LisCADv6.2, Log2.1, Proshake, Curve expert 1.3, UIUC developed software, MATLAB and a designed computer code by authors namely “Abbas Converter”, the response spectra, computed motion and some related parameters for the selected area were evaluated and compared. As a result of the limitation in software applicability, none of the above softwares can reply to all the requested parameters alone. As it is known,

Table 1. List of some happened events with M≥4.5 in the region *(www.ngdir.ir).*

Date	M_b	M_s	Long.	Lat.	Reference	Event time
2002/04/07	---	4.5	45.261	38.384	NEIC	22:50:31
2001/06/12	4.6	4	47.262	38.995	ISC	01:46:52
1999/08/19	4.5	---	46.42	38.417	ISC	04:33:19
1998/11/23	4.5	4	45.136	38.352	ISC	11:11:38
1996/04/22	4.9	---	47.332	39.186	ISC	14:42:37
1993/03/15	4.7	4	45.826	38.125	ISC	15:32:38
1989/12/03	4.8	4	45.351	38.442	ISC	07:39:11
1989/12/02	4.5	---	45.425	38.453	ISC	04:51:59
1988/01/07	4.8	---	45.528	38.478	ISC	10:59:39
1986/07/10	4.6	---	45.221	38.371	ISC	18:57:17
1984/08/24	4.9	---	45.952	38.496	ISC	11:31:41
1984/03/25	4.6	---	45.28	38.245	ISC	02:44:58
1981/05/24	4.5	4	45.464	38.412	ISC	22:07:08
1981/05/24	4.5	4	45.43	38.521	ISC	21:12:25
1980/10/10	4.8	4	45.908	38.402	ISC	11:09:53
1979/11/21	4.6	4	47.229	38.191	ISC	15:36:05
1971/02/11	4.5	---	47.123	38.345	ISC	01:41:30
1970/10/29	4.5	---	45.47	38.43	ISC	08:49:32
1965/02/10	5	---	47.09	37.66	ISC	16:09:54
1963/12/31	4.5	---	45.3	38.4	NEIC	15:18:08
1954/10/22	---	5	45.8	38.9	KAR	22:47:29
1931/07/04	---	5	45.6	38	KAR	21:00:50
1931/05/12	---	5	46.3	38.8	KAR	10:25:13
1930/05/29	---	6	45.5	37.5	KAR	17:14:55
1930/05/23	---	5	45.5	37.5	KAR	09:48:20
1930/05/08	---	5	45.5	37.5	KAR	15:05:21
1928/03/24	---	5	47.3	37.8	KAR	10:53:16

if any of them uses a specific format different from others, their data would not be applicable to one another and to this study. For this reason, the authors were forced to produce a computer program to generate the new motions for them and convert the primary input data of the mentioned softwares to each of them. This is the main reason for designing the “Abbas Converter”.

LOCAL GEOLOGY AND GROUND RESPONSE

The local soil conditions have profound influence on ground response during earthquakes. The recent destructive earthquakes (Mexico City, 1985; Loma Prieta, 1989; Northridge, 1994; Kobe, 1995; Kocaeli, 1999; Colombia, 1999; Bam, 2003; etc.) have brought additional evidence on the importance of site effect on ground motions. This problem is commonly referred to as a site specific response analysis or soil amplification study. Accounting for such effects has therefore gained critical importance in seismic regulations, land use planning and seismic design of critical facilities; and for obtaining this aim, the acceleration response spectra are mainly used to predict the effects of earthquake magnitudes on the relative frequency content of ground-bedrock motions. Figure 2 shows some of the soil conditions and local geological features affecting the ground response.

A number of techniques are available for ground response analyses which differ in the simplifying assumptions that are made, in the representation of stress-strain relations of soil and in the methods used to integrate the equation of motion (Arsalan and Siyahi, 2006; Park and Hashash, 2004). The development of practice needs for available response analysis methods and the increasing knowledge about the basic soil behavior under cyclic loading were derived from field observations and laboratory testing. Therefore, empirical procedures have been developed to estimate site effects, but are limited in applications. Site response analysis is commonly performed to estimate and characterize site effects by solving the dynamic equations of motion via an idealized soil profile. There are two main numerical methods for its solving, which are namely equivalent linear analysis method (frequency domain solution) and nonlinear analysis method (time domain solution).

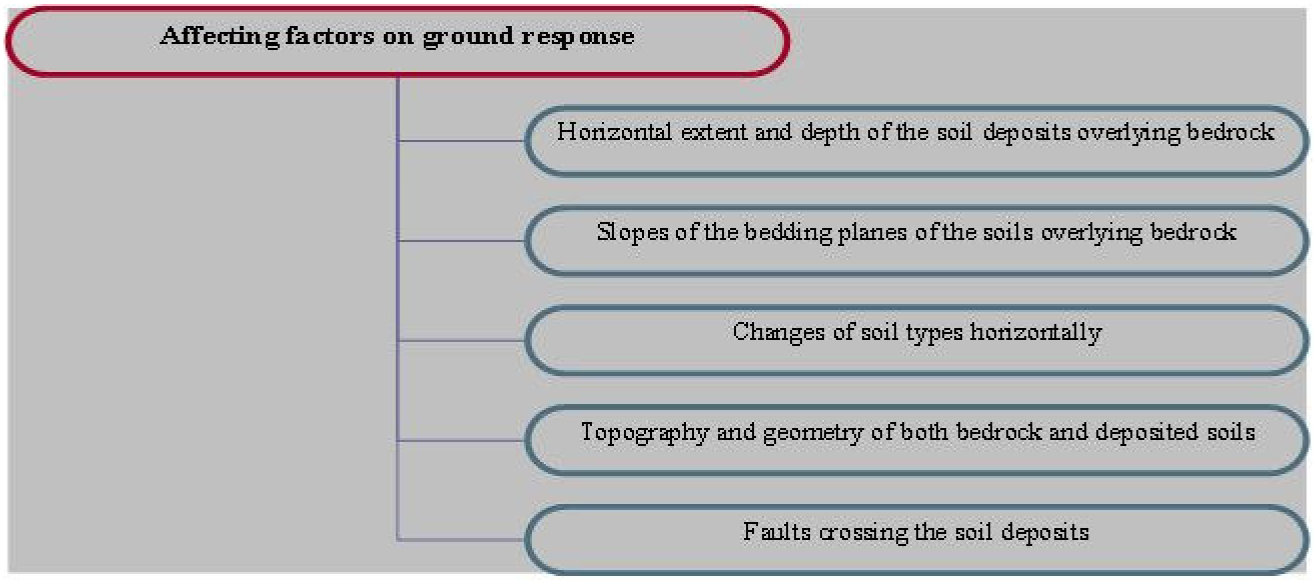

Figure 2. Effective factors on ground response.

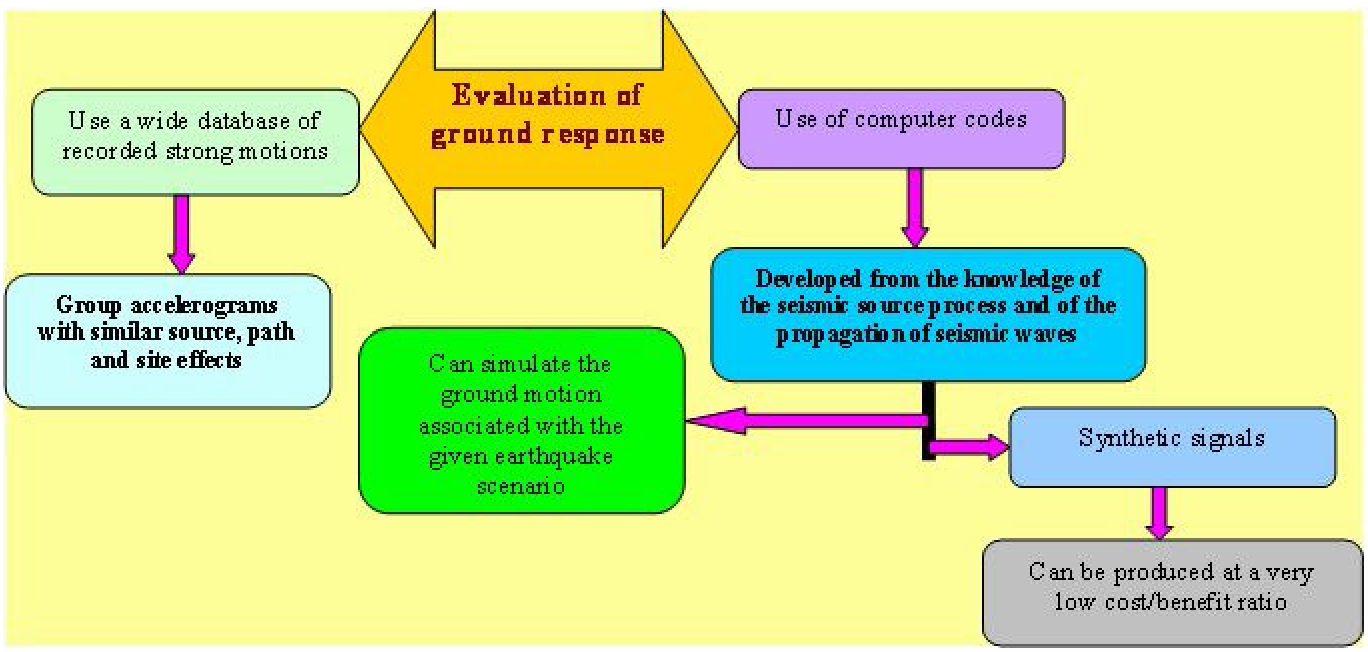

Figure 3. Alternatives for evaluation of ground response (Borja et al., 1999; Elgamal et al., 1996).

Yoshida (1994), Huang et al. (2001) and Yoshida and Iai (1998) showed that the equivalent linear analysis shows larger peak acceleration because the method computes and takes into account the acceleration in a high frequency large range. The nonlinearity of soil behavior is known very well, thus, most reasonable approaches to provide reasonable estimates of site response are a very challenging area in geotechnical earthquake engineering.

The strain vibration during loading is significant and can not be approximated by a representative strain throughout the duration of shaking; thus, on the basis of Figure 3, evaluation of ground response is one of the most crucial problems encountered in geotechnical earthquake analysis. So, the basic problem associated with the study of seismic hazard is the determination of the seismic ground motion at a given site, due to an

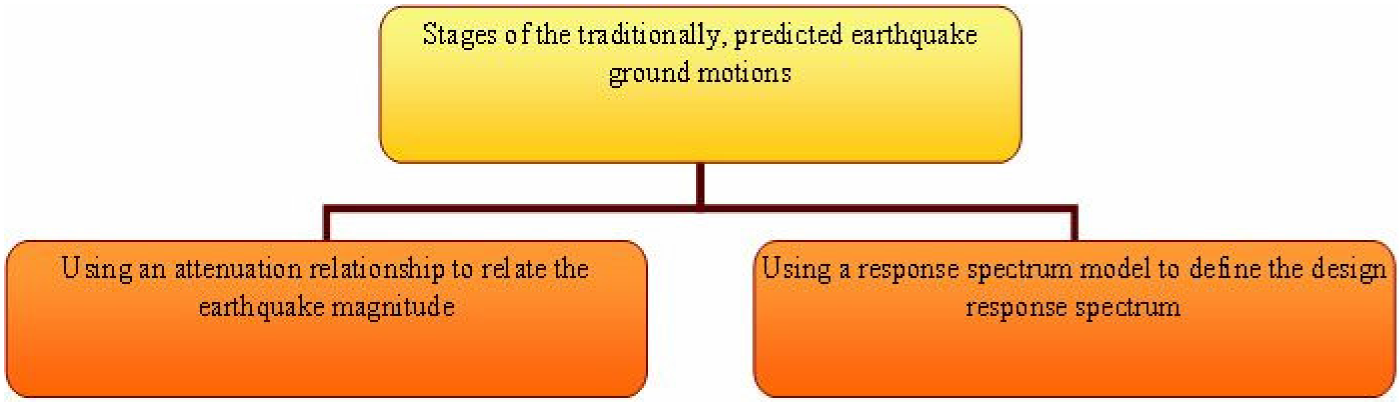

Figure 4. Stages of prediction of the ground motion (Boominathan, 2004; Lam et al., 2000).

earthquake incite, with a given intensity and epicentral distance.

The objective of a site response analysis is to estimate the ground shaking during an earthquake, in which shaking at sites does not include effects caused by proximity to structures or topographic features, for a specific hazard level and set of site conditions. One or more design earthquake events with representative earthquake record(s), is an idealization of the soil-rock system at the site of interest and a scheme to generate response solutions to simplified assumed wave fields, which are the requisite components for a site response analysis. Normally, the ground response is presented in terms of either response spectra or the variation of acceleration or velocity with time. Figure 4 shows the stages of traditionally, predicted earthquake ground motions.

Effective factors on site effect, response and amplification

The importance of site effect on seismic motion has been realized since the 1920s and the quantitative studies have been conducted using strong motion array data after the 1970s. During earthquakes, the ground motion parameters such as motion amplitude, frequency content and duration of the ground motion changes as the seismic waves propagate through overlying soil and reach the ground surface. The phenomenon, where in local soils, acts as a filter and modifies the ground motion characteristics, is known as 'soil amplification' which is defined as the ratio of the peak horizontal acceleration at the ground surface to the peak at the bedrock (Anbazhagan 2007). Site amplification, due to soil conditions and the resulting damage to built environment, was amply demonstrated by past earthquakes (Elgamal et al., 2005). As seismic waves travel from bedrock to the surface, certain characteristics of the waves, such as amplitude and frequency content is changed as they pass through the soil deposits. Site specific ground response analysis aims at determining this effect of local soil conditions on amplification of seismic waves, and hence, estimating the ground response spectra for future design purposes. As shown in Figure 5, earthquake ground motion can be significantly amplified by 'superficial deposits'. Even though seismic waves generally travel tens of kilometers of rock and less than 100 m of soil, the soil plays a very important role in determining the characteristics of ground motion (Kramer, 1996). Therefore, understanding the site response of geological materials under seismic loading is an important element in developing a well-established constitutive model. Topographic and basin effects, Liquefaction, Ground failure and Structural deficiencies are potentially factors contributing to damage. The amplification of ground motion due to local site conditions plays an important part in increasing seismic damage (Rodriguez et al. 2000).

Site response is also a function of profile depth; thus, ignoring profile depth may have a detrimental effect in ground motion prediction. Also, it can be introduced into the most current attenuation relationships (Park and Hashash, 2004; Rodriguez et al., 2000). "Dynamic stiffness, depth, impedance ratio between the soil deposit and underlying bedrock, the material damping of the soil deposits and the nonlinear response of a soft potentially liquefiable soil deposits" are important factors in seismic site response.

The effect of nonlinearity is largely a function of soil type (Vucetic, 1990; Vucetic and Dobroy, 1991; Sitharam et al., 2004). 'Cementation and geologic age' may also affect the nonlinear behavior of soils (Field et al., 1997). To account partially for these factors, a site classification scheme should include the nonlinear behavior of soil and the measurement of the dynamic stiffness of the site and depth of the deposit (Rodriguez et al. 2000). The response of a soil deposit is dependent on 'frequency of the base motion, the geometry and the material properties of the soil layer above the bedrock'.

To account partially for these factors, a site

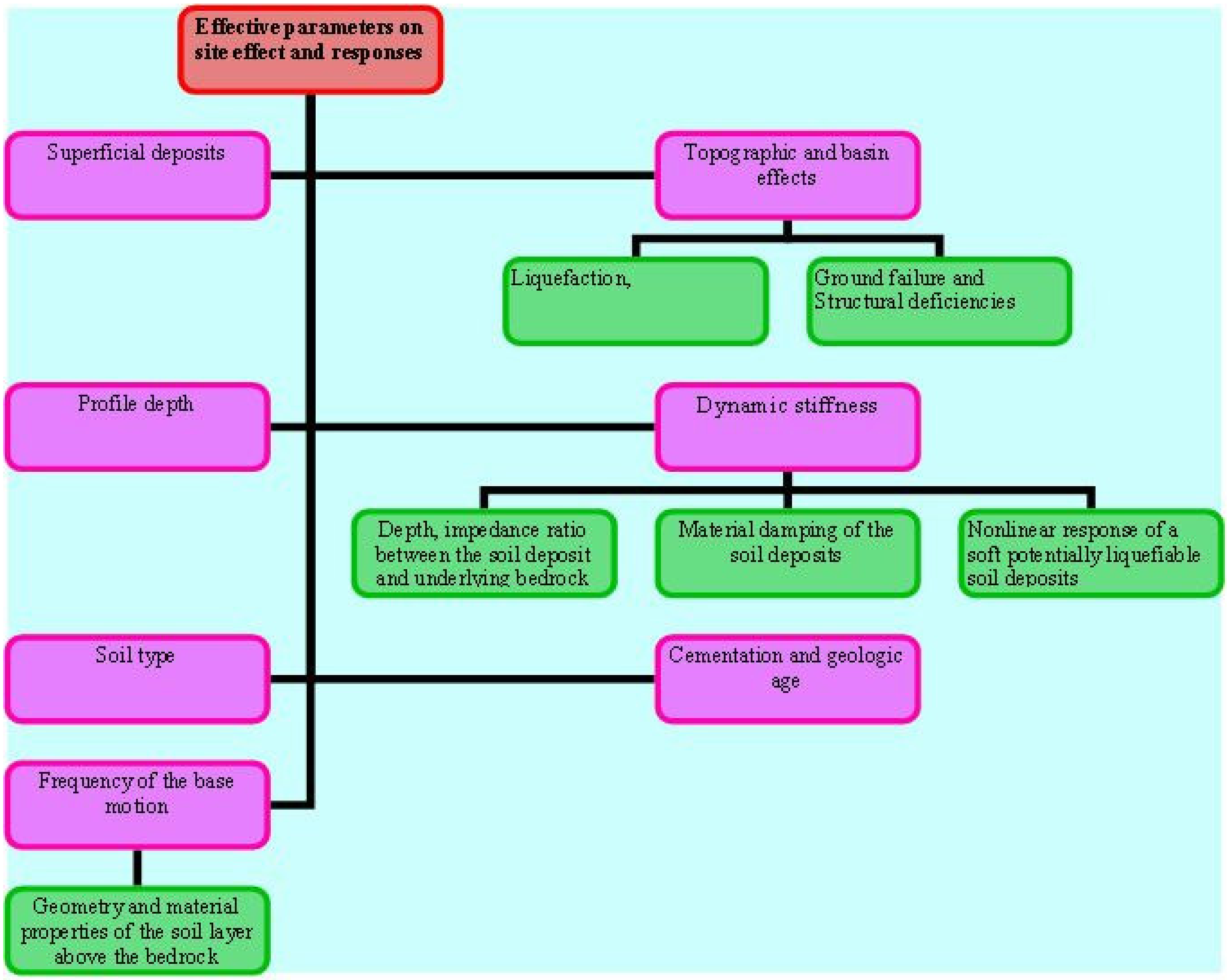

Figure 5. Effective factors on seismic site effect and site response.

classification scheme should include the nonlinear behavior of soil and the measurement of the dynamic stiffness of the site and depth of the deposit (Elgamal et al., 2005). There have been many researches on site response analysis of shaking ground under earthquake loading and excitation.

Schnabel et al. (1972) proposed an equivalent linear approach that is widely used for site response analysis. Seed et al. (1976), base on the statistical study of 147 recordings from western U.S earthquakes of about 6.5 magnitude, developed peak acceleration attenuation relationships for different site conditions. Idriss (1990) developed an empirical correlation between the peak acceleration of rock outcrop and soft soil. The relation is based on recordings from Mexico City (1985) and Loma Prieta (1989). Kramer (1996) developed a nonlinear approach, by which a nonlinear inelastic stress-strain relationship was followed in a set of small incrementally linear steps. Field et al. (1997), in the view of geotechnical engineers that was based largely on laboratory studies, said that Hook's law (linear elasticity) breaks down at larger strains causing a reduced (nonlinear) amplification. Borja et al. (1999) fully developed a nonlinear finite element model to investigate the impact of hysteretic and viscous material behavior on the down-hole motion recorded by an array at large scale seismic test site in Lotung, Taiwan, during the earthquake of 20 May, 1986. Rodriguez et al. (2001) proposed an empirical geotechnical seismic site response procedure that accounts for the nonlinear stress-strain response of earth materials under earthquake loading.

LINEAR AND NONLINEAR BEHAVIOR

The nonlinearity of soil behavior is known very well thus, most reasonable approaches to provide reasonable estimates of site response are very challenging area in geotechnical earthquake engineering. The main reason for using the linear approach is because the method is computationally convenient and provides reasonable results for some practical cases (Kramer, 1996). The nonlinearity of soil stress-strain behavior for dynamic analysis means that the shear modulus of the soil is constantly changing. Both time and frequency domain analyses are used to account for the nonlinear effects in site response problems. Nonlinear and equivalent linear methods are utilized respectively in the time and frequency domain for the 1D analysis of shear wave propagation in layered soil media. When compared with earthquake observation, nonlinear analysis is shown to agree with the observed record better than the equivalent linear analysis (Arsalan and Siyahi, 2006).

The nonlinear behavior of soils is the most important factor in ground motion propagation and should be accounted for when soil shearing strains are expected to exceed the linear threshold strain. In site response analysis, soil properties including shear modulus and cyclic soil behavior are required. Shear modulus is estimated using field tests such as seismic down-hole or cross-hole tests. Cyclic soil behavior is characterized using laboratory tests such as resonant column, cyclic triaxial or simple shear tests. The maximum shear modulus is defined as G_{max} and corresponds to the initial shear modulus. The slope of stress-strain curve at a particular strain is a tangent shear modulus (G_{tan}). The secant shear modulus (G_{sec}) is the average shear modulus for a given load cycle. *In situ* measurement of V_s using geophysical methods is the best method for measuring the G_{max} (Rolling et al. 1998). Geophysical methods are based on the fact that the velocity of propagation of a wave in an elastic body is a function of the modulus of elasticity, Poisson ratio and density of material (Hvorslev, 1949).

On the ground surface at strain levels less than 0.001%, G_{max} can be determined as $G_{\max} = \rho V_s^{\,2}$ from the measured V_s profile by assuming the density (ρ) and G_{max} can also be estimated directly from N values in the field as $G_{\max} = aN^b$ (a, b: correlation coefficients). Several correlations are reported between V_s and N values measured in the field which is often expressed in the form of $V_s = AN^B$ (A, B: constant parameters and are often accompanied by a correlation coefficient R). Usually the trend observed is that if A increases B decreases for the same type soil (Ohasaki and Iwasaki, 1973; Imai, 1977; Ohta and Goto, 1978; Imai and Tonouchi, 1982). Some of the V_s- N correlations were reported by Hanumantharao and Ramana (2008).

By considering a uniform soil layer lying on an elastic layer of rock that extends to an infinite depth in which the subscripts *s* and *r* refer to soil and rock, the horizontal displacement due to vertically propagated harmonic S wave in each material can be written as:

$$u_s(Z_s,t) = A_s e^{i(\omega t + K_s^* Z_s)} + B_s e^{i(\omega t - K_s^* Z_s)} \quad \text{.......... (1)}$$

$$u_r(Z_r,t) = A_r e^{i(\omega t + K_r^* Z_r)} + B_s e^{i(\omega t - K_r^* Z_r)} \quad \text{........... (2)}$$

u: displacement, ω: circular frequency of the harmonic wave, k^*: complex wave number

No shear stress can exist at the ground surface (z_s= 0), so:

$$\tau_{(0,t)} = G_s^{\,*} \gamma_{(0,t)} = G_s^{\,*} \frac{\partial u_s(0,t)}{\partial z_s} = 0 \quad \text{..................... (3)}$$

where G_s^*= G (1+2iξ) is the complex shear modulus of the soil. Schnabel et al. (1972) explained that within a given layer (layer *j*), the horizontal displacements for two motions (A and B) may be given as:

$$u_r(z_i,t) = (A_i e^{ik_i^* z_i} + B_i e^{-ik_i^* z_i}) e^{i\omega t} \quad \text{...................... (4)}$$

Thus, at the boundary between layer *j* and *j+1*, compatibility of displacements requires that:

$$A_{j+1} + B_{j+1} = A_j e^{ik_j^* h_j} + B_j e^{-ik_j^* h_j} \quad \text{........................ (5)}$$

Continuity of shear stresses requires that:

$$A_{j+1} + B_{j+1} = \frac{G_j^{\,*} k_j^{\,*}}{G_{j+1}^{\,*} k_{j+1}^{\,*}} (A_j e^{ik_j^* h_j} - B_j e^{-ik_j^* h_j}) \quad \text{...... (6)}$$

The effective shear strain of equivalent linear analysis is computed as:

$$\gamma_{eff} = R_\gamma \gamma_{\max} \quad \text{.......................... (7)}$$

$$R_\gamma = \frac{M-1}{10} \quad \text{............................. .... (8)}$$

γ_{max}: maximum shear strain in the layer, R_γ: strain reduction factor, M: magnitude of earthquake.

The motion at any layer can be easily computed from the motion at any other layer (for example, input motion imposed at the bottom of the soil column) using the transfer function that relates displacement amplitude at layer *i* to that in layer *j*:

$$F_{ij}(\omega) = \frac{|u_i|}{|u_j|} = \frac{a_i(\omega) + b_i(\omega)}{a_j(\omega) + b_j(\omega)} \quad \text{........................ (9)}$$

The nonlinearity of soil stress-strain behavior for dynamic analysis means that the shear modulus of the soil is constantly changing. Nonlinear and equivalent linear methods are utilized respectively in the time and frequency domain for the 1D analysis of shear wave propagation in layered soil media. When compared with earthquake observation, the nonlinear analysis is shown to agree with the observed record better than the equivalent linear analysis (Arsalan and Siyahi, 2006).

The nonlinear hyperbolic model used in this paper was developed by Konder and Zelasko (1963) to model the stress-strain soil behavior of soils subjected to constant rate of loading. The hyperbolic equation is defined as:

$$\tau = \frac{G_{mo}\gamma}{1+(\frac{G_{mo}}{\tau_{mo}}\gamma)} = \frac{G_{mo}\gamma}{1+(\frac{\gamma}{\gamma_r})} \quad \text{..............(10)}$$

τ: shear stress, γ: shear strain, G_{mo}: initial shear modulus, τ_{mo}: shear strength, $\gamma_r = \tau_{mo}/ G_{mo}$: reference shear strain. The reference shear strain is the strain at which failure would occur if the soil were to behave elastically. It has been considered to be a constant material by Hardin and Drnevich (1972). The reference strain can also be represented as the function of the initial tangent modulus and the undrained shear strength in clays (Mersi et al., 1981).

1D site response analysis is commonly performed to account for local site effects on ground motion propagation during an earthquake (Idriss, 1990; Kramer, 1996). The solution of the wave propagation equation is performed in either frequency or time domain (Park and Hashash, 2004).

The soil medium is divided into sub layers with absolute displacement u_j, defined at j^{th} sub layer interface and shear stress τ_j, defined at the mid points of each interface. As Kramer (1996) explained, the response of soil deposit under dynamic loading is governed by the equation of motion as follows:

$$\frac{\partial\tau}{\partial z} = \rho\frac{\partial^2 u}{\Delta t^2} \quad \text{.................................... (11)}$$

The differentiation for a soil divided to N sub layers of thickness Δz and processing for the small time increment Δt is computed by using finite difference method as follows:

$$\frac{\partial\tau}{\partial z} = \frac{(\tau_{i+1}-\tau_{i-1})}{\Delta z} \quad \text{.................... (12)}$$

$$\frac{\partial^2 u}{\partial t^2} = \frac{(\dot{u}_{i,t+\Delta t}-\dot{u}_{i,t})}{\Delta t} \quad \text{..................... (13)}$$

$\dot{u} = \frac{\partial u}{\partial t}$: Velocity of the motion, $\frac{\partial^2 u}{\partial t^2} = \frac{\partial \dot{u}}{\partial t}$: acceleration

The result of the combination for equations 11, 12 and 13 will be:

$$\frac{(\tau_{i+1}-\tau_{i-1})}{\Delta z} = \rho\frac{\dot{u}_{i,t+\Delta t}-\dot{u}_{i,t}}{\Delta t} \quad \text{.................. (14)}$$

And it can be simplified as:

$$\dot{u}_{i,t+\Delta t} = \dot{u}_{i,t} + \frac{\Delta t}{\rho\Delta z}(\tau_{i+1,t}-\tau_{i,t}) \quad \text{............ (15)}$$

As mentioned above for the soil surface, the shear stress is equal to zero and the boundary condition for each sub layer must be satisfied. For soil rock boundaries Joyner and Chen (1975) proposed the following equation for soil rock boundaries:

$$\tau_{r,t} = \rho_r V_{sr}\left[2\dot{u}_r(t+\Delta t) - \dot{u}_{N+1,t+\Delta t}\right] \quad \text{............. (16)}$$

By using equations 15 and 16, the boundary conditions are satisfied. Kramer (1996) proposed the shear for each layer as shown:

$$\gamma_{i,t} = \frac{\partial u_{i,t}}{\partial z} \approx \frac{(u_{i+1}-u_{i,t})}{\Delta z} \quad \text{.................. (17)}$$

As the above mentioned equations show, the shear stress is computed by using current shear strain and stress-strain history ($\tau_{i,t}=G_i\ \gamma_{i,t}$). Thus, the proposed method satisfies the nonlinear and inelastic behavior of soil under earthquake excitation.

For strong vibrations (medium and large earthquakes), the linear elastic solution is no longer valid because the soil behavior is inelastic, non-linear and strain dependent. Equivalent linear analysis, performed in the frequency domain, has been developed to approximate the nonlinear behavior of soil. The frequency domain solution of wave propagation provides the exact solution when the soil response is linear. The equivalent linear method approximates nonlinear behavior by incorporating a shear strain dependent shear modulus and damping soil curves (Park and Hashash, 2004).

ANALYSIS METHOD

As indicated in Figure 6 for effective site parameters and by referring to Figure 7, the analysis method steps are as follows:

1. Characterization of site based on field investigation

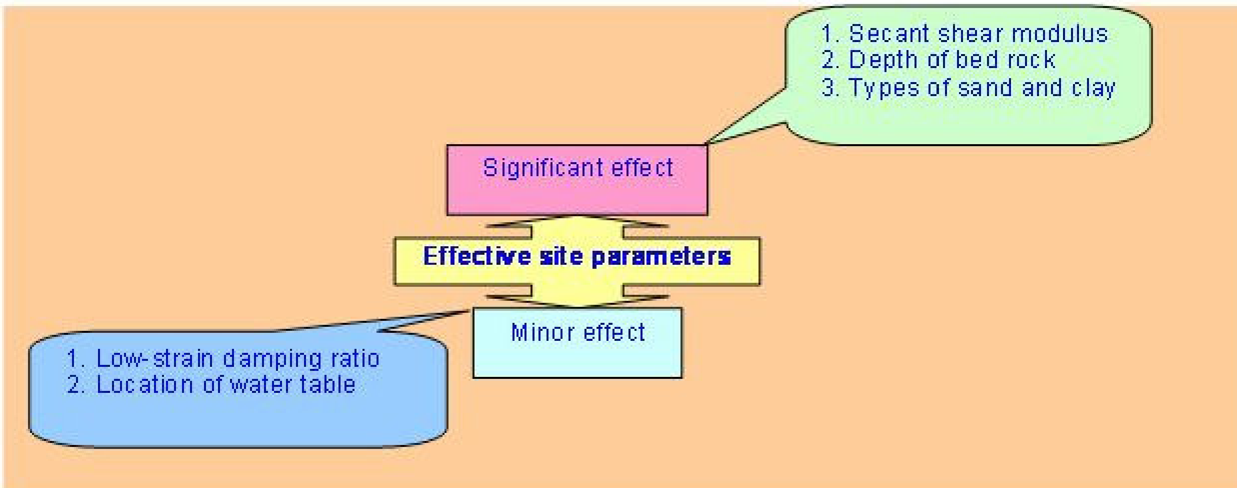

Figure 6. Classification of effective site parameters (Seed and Idriss, 1970; Joyner and Chen, 1975; Hwang and Lee, 1991).

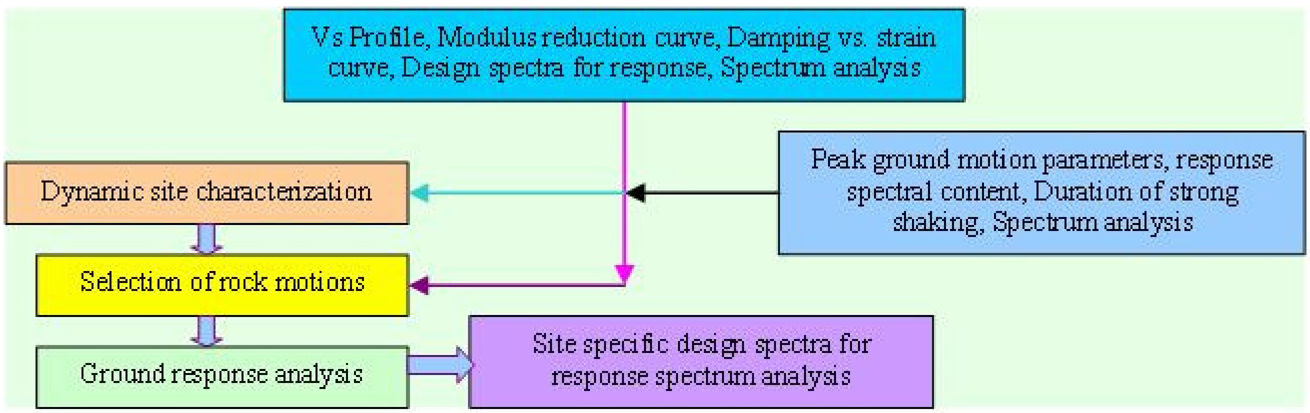

Figure 7. Site specific ground response analysis.

and laboratory test.

2. Elect and apply the rock motion on soil profile column, associated with seismotectonic structure, to represent the effect of motion for the site on the soil profile.

3. Analysis of site response, development and improvement of site surface response spectra.

The main reason for designing the “Abbas converter” was to generate new data, information and motions corresponding to seismotectonic of the selected area and convert the primary input of softwares to the other. This produces code work and installs quickly. Also, it operates as a logic connecter link between the used softwares, which can generate the input data that correspond to the defined format for the used softwares. Its output results can easily be exported to the other used software in this study. This designed software make and render easy the study more than the previous ones have done. With it also, the authors could enter the recorded data with different format as an input and take the defined format for the used softwares. As shown in Figure 8, this code has a graphical user interface, which allows the user to select the required item. In this code, the user has two choices. The authors have placed the recorded earthquake of Iran as the sample input files (Abbaszadeh Shahri et al., 2009). Also, it is possible to select the recorded earthquake of the other countries by clicking in ‘input earthquake data’ and browsing the desired files. After the selection of the input file, this code will generate several new input motions that can apply for the other softwares. In addition, this code enables the presentation of the motion properties before exporting it to the software. In the ‘geotechnical model’, the user can simulate the idealized soil profile by the obtained data from field and laboratory tests and the code will compute the soil characteristics by a consideration of ground water table more than it can present the type of geotechnical category of the site according to the defined procedure.

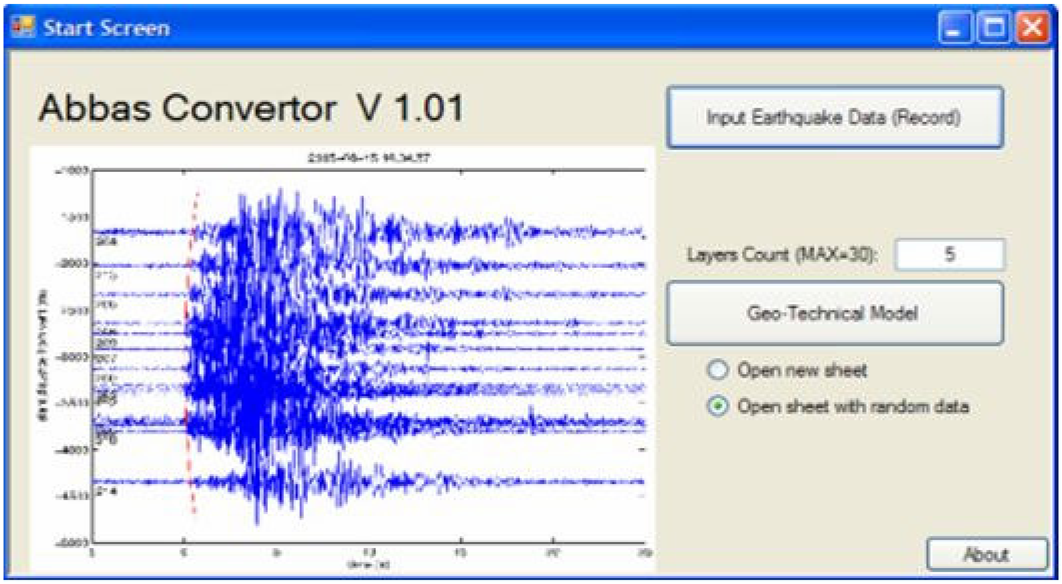

Figure 8. Start screen of “Abbas Converter”.

The detailed proposed flowchart of this study and also the ability and capability of “Abbas converter” is shown in Figure 9. By referring to Figure 10, this procedure indicated that the designed computer code can work in different conditions. With this method, the authors could enter new and different format of data as input and take the defined format for the software. For soil properties modeling in nonlinear time domain analysis in this study, the standard hyperbolic model with elastic and rigid half space bedrock, flexible time control, maximum strain increment (about 0.005) and damping matrix defined with modes and frequency were executed. In addition, it is important to note that, the accuracy of time domain solution depends on the time steps.

The L component of Ardabil event at bedrock was applied to each borehole location based on the hypocentral distance calculated for each of them in the Miyaneh region to study the site response as shown in Figures 11 - 13. The recorded data were picked up from BHRC web site of Iran and were drawn with seismosignal In order to obtain reliable information and accurate data regarding the structural pattern of the subsurface soil, among a total of 28 drilled bore holes, 10 borelogs were carefully evaluated, but the results of two of them with a minimum of 40m depth (namely BH1 and BH10) were selected and presented. In these sites, water table was filled up to the depth of 9m for BH10 and 5 m for BH1 at the time of drilling operation. Soil profile as shown in Tables 2 and 3 for comparison, must be created and modified. V_s of surface sediments were investigated in correlation with geotechnical properties determined by laboratory testing and in addition, lithofacies based on detailed core investigation were taken into account by the correlation analysis. N values obtained by *in situ* field measurement SPT, bulk densities, solidities and mean grain size measured by the standard soil test and V_s were correlated to N values to obtain the empirical relationship between them. In view of this, no attempts were made for developing the regression correlation based on the entire dataset and N values from locations where tests were conducted. Thus, for this study, 180 pairs of N value and V_s were applied and a formula which explained V_s as a function of N value was determined for the selected area as shown in Table 4.

From the wave propagation theory, it is clear that the ground motion amplitude depends on the density and V_s of subsurface material. Usually, *in situ* density has relatively smaller variation with depth and thus, the V_s is the logical choice for representing site conditions. The time histories as input motions are applied on the bottom of the profile and assigned to measure the hypothetical rock outcrop at the site rather than directly at the base of the soil profile. For two different conditions, the input and computed motions were presented in Figure 14. Site response analysis is conducted for the modeled, idealized, developed and improved soil profiles to determine and calculate ground motions at the site surface. Response spectra of the site surface motion were computed for the various analyses made as shown in Figure 15. This is because the knowledge of motions is based on recording at rock outcrops. Unless the rock is rigid, the motions at the base of the soil profile will differ from those of the outcrop more than the amplification spectrum can be obtained between the first and top layer. These results are given in Figures 16 and 17. Stress-strain time histories were presented in Figures 18 and 19 for elastic and rigid half space.

By comparison of the above figures, the following results can be under took and summarized in Tables 5 and 6.

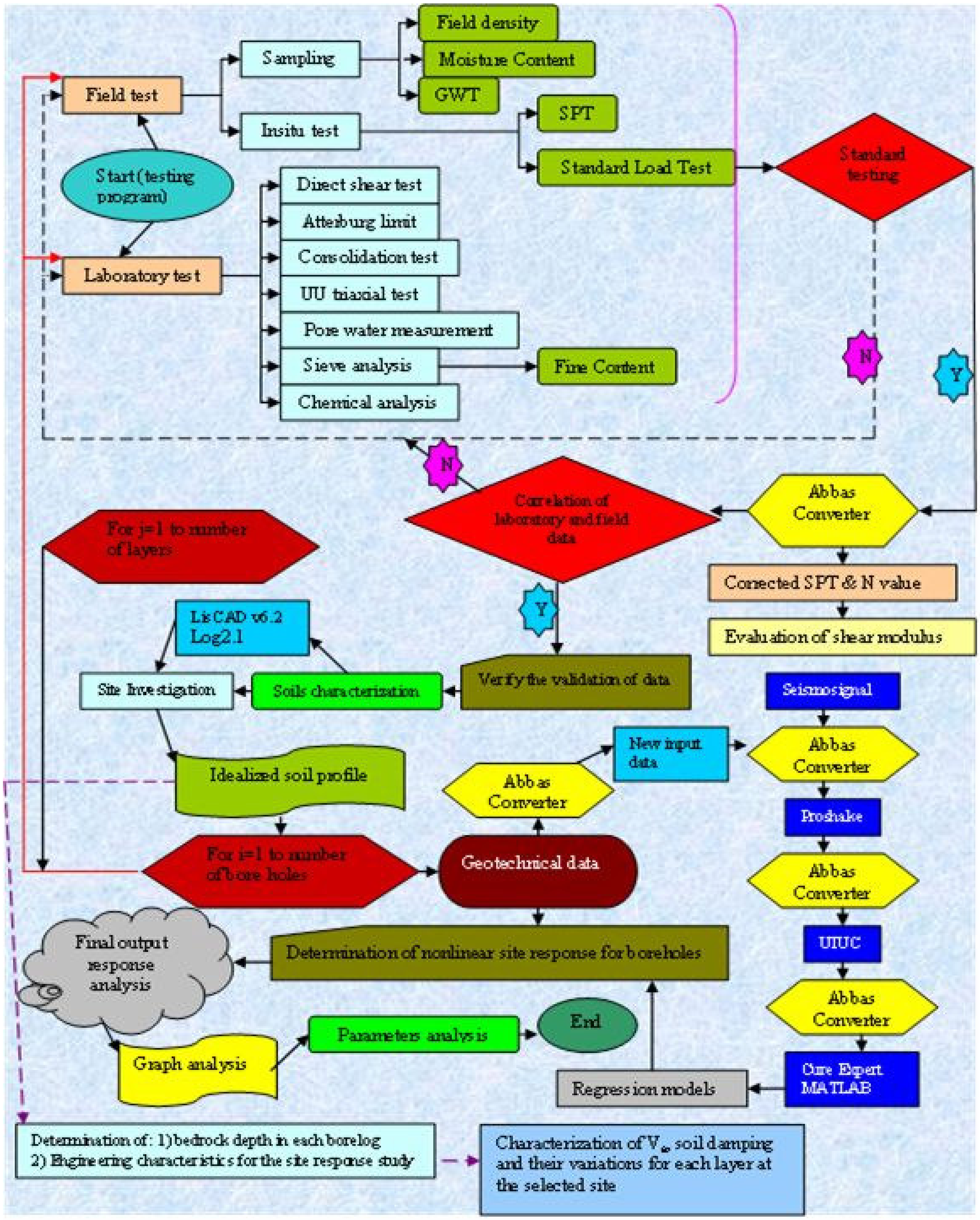

Figure 9. Proposed flowchart for this study by the authors.

DISCUSSION AND CONCLUSION

This study tried to conduct a meaningful site response and amplification study. The difficulties/uncertainties in choosing an input ground motion are discussed, and the various methods currently available for the site response study are summarized. A case study on ground response analysis of a site in Miyaneh City in Azarbayjan-Sharghi province of Iran, during the Ardabil earthquake, is presented. The study shows that the measurement and prediction of ground vibration due to strong motions have demonstrated the predominant role of site effects in the

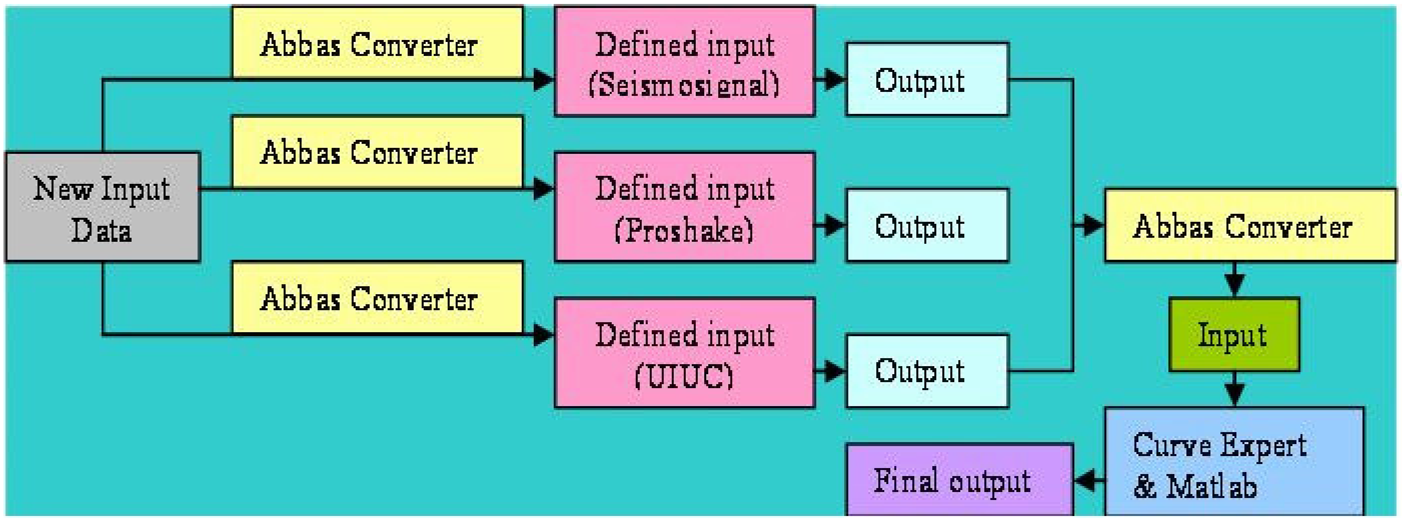

Figure 10. Ability of "Abbas Converter" to install in parallel condition.

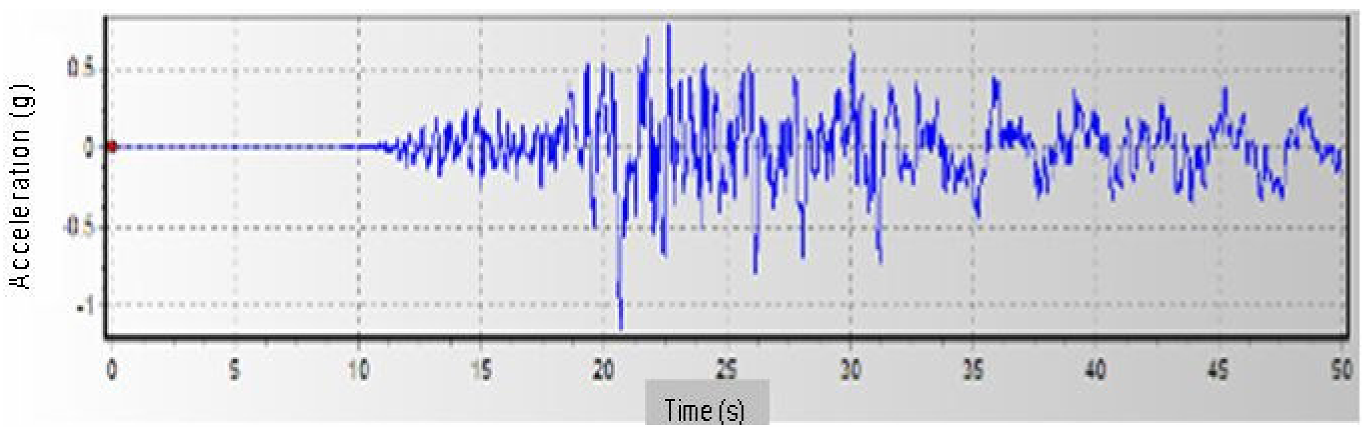

Figure 11. L Component of Ardabil event, PGA = 1.1447g at t = 20.66s (Abbas Converter and seismosignal).

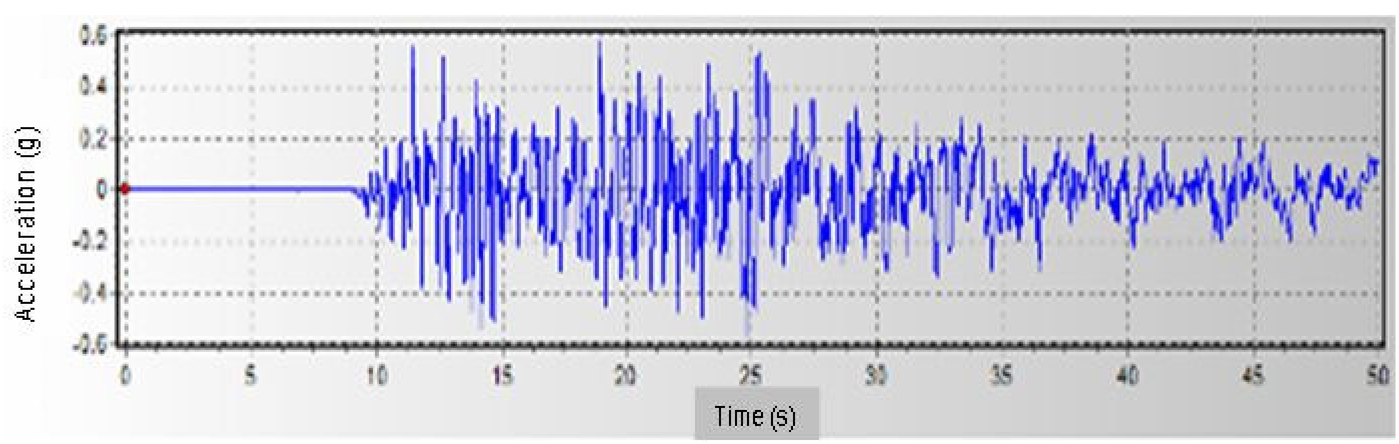

Figure 12. V Component of Ardabil event, PGA = 0.584412g at t = 18.94s (Abbas Converter and seismosignal).

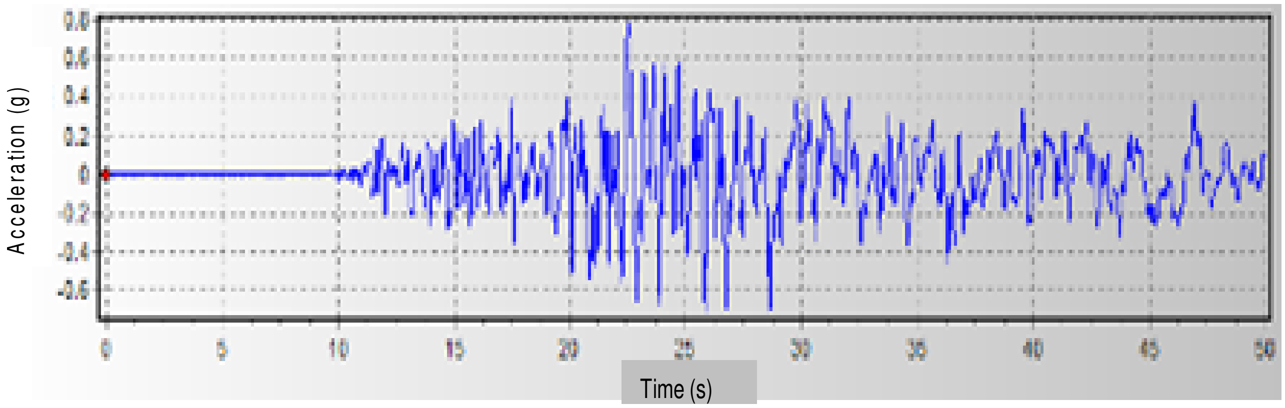

Figure 13. T Component of Ardabil event, PGA = 0.778382g at t = 22.5s (Abbas Converter and seismosignal).

Table 2. Soil profile of BH10.

Soil type	$Depth_{(m)}$	$Thickness_{(m)}$	$\gamma_{(gr/cm3)}$	SPT	PI	$Vs_{(m/s)}$
CL	1.5	1.5	1.55	37	23	244.926
SC	3.5	2	1.53	46	17	270.466
CL	12	8.5	1.62	55	20	292.04
SM	14.5	2.5	1.7	65	---	319.19
CL	16.5	2	1.73	59	22	300.618
SM	18.5	2	1.71	73	30	328.326
CL	20.5	2	1.68	60	23	305.455
CH	24.5	4	1.73	58	18	298.506
MH	26.5	2	1.71	72	27	326.44
CL	30.5	4	1.81	54	25	289.849
CH	32.5	2	1.71	61	27	301.1
CL	44.5	12	1.84	73	21	368.326

Bedrock γ (2.0gr/cm^3), Vs = 1016.125 m/s.

Table 3. Soil profile of BH1.

Soil type	$Depth_{(m)}$	$Thickness_{(m)}$	$\gamma_{(gr/cm3)}$	SPT	PI	$Vs_{(m/s)}$
SC	1.5	1.5	1.53	29	12	230.95
GP	3.5	2	1.77	43	---	265.603
SP-SM	7.5	4	1.8	88	---	368.022
CL	31.5	24	1.78	53	14	287.629
CL	33.5	2	1.82	78	10	337.499
GC	36	2.5	1.85	66	12	319.1
CL	40	4	1.9	70	17	322.651

Bedrock γ (2.21gr/cm^3), Vs=1214.2 m/s.

response of infrastructure during a seismic event. Site response analysis is usually the first step of seismic geotechnical study and authors have been trying to find a practical and appropriate solution for ground response analysis under earthquake forces for the selected site.

The practice of earthquake geotechnical engineering involves the identification and modeling of the rupture mechanism at the source of an earthquake, evaluating the propagation of waves through the earth to the top of bed rock, determining the effect of local soil profile and thus, to develop a hazard map indicating the vulnerability of the area to the potential seismic hazard. The geotechnical engineer is responsible for providing the structural engineer with appropriate site-specific design

Table 4. Correlation results of Vs- N for selected region (curve expert 1.3 and MATLAB).

Model	a	b	R	S
$V_s = aN^b$	54.792	0.42007	0.9650	9.7144
$V_s = a+bN$	160.1653	2.39785	0.9699	9.019(X)
$V_s = a+N^b$	177.903	1.18007	0.9688	9.176
$V_s = a+b^N$	252.7499	1.0603	0.7839	23.009
$V_s = ab^N$	183.065	1.00837	0.9644	9.79
$V_s = ae^{bN}$	183.065	0.00834	0.9644	9.7902
$V_s = a+bLnN$	-161.859	114.447	0.9526	11.2747

a, b: Constant parameters, R: Correlation coefficient, S: Standard error.

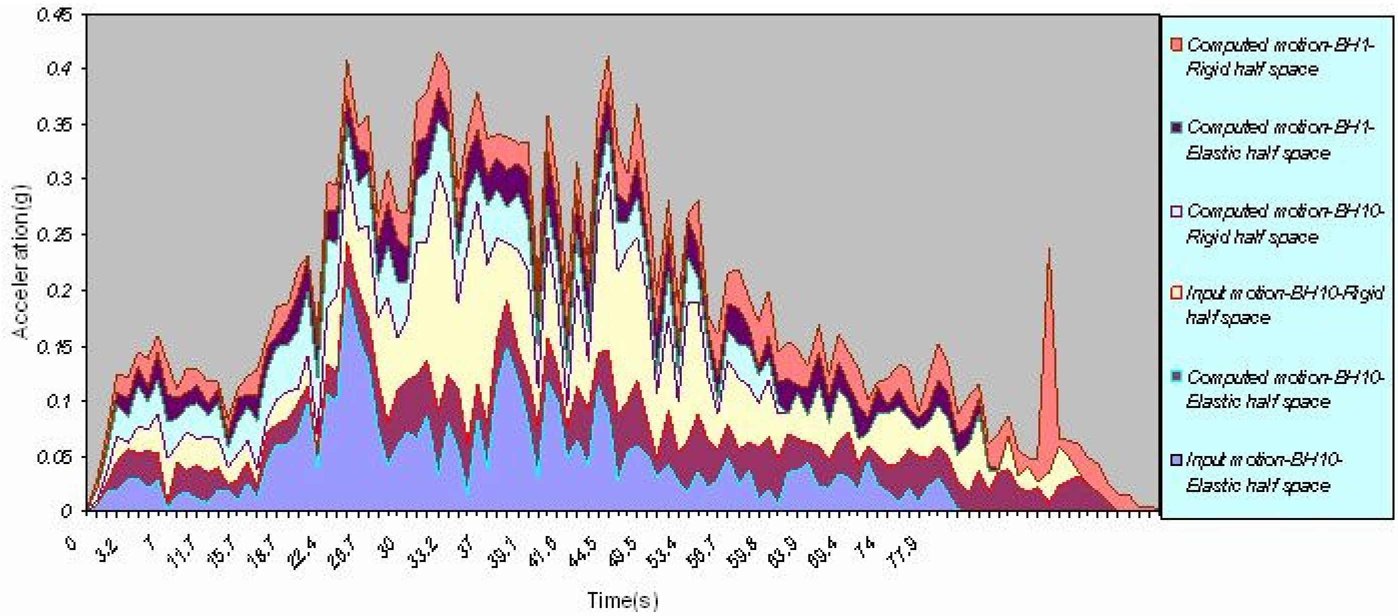

Figure 14. Comparison between the input and computed motion in elastic and rigid half space by 5% damping ("Abbas Converter", Excel and Matlab).

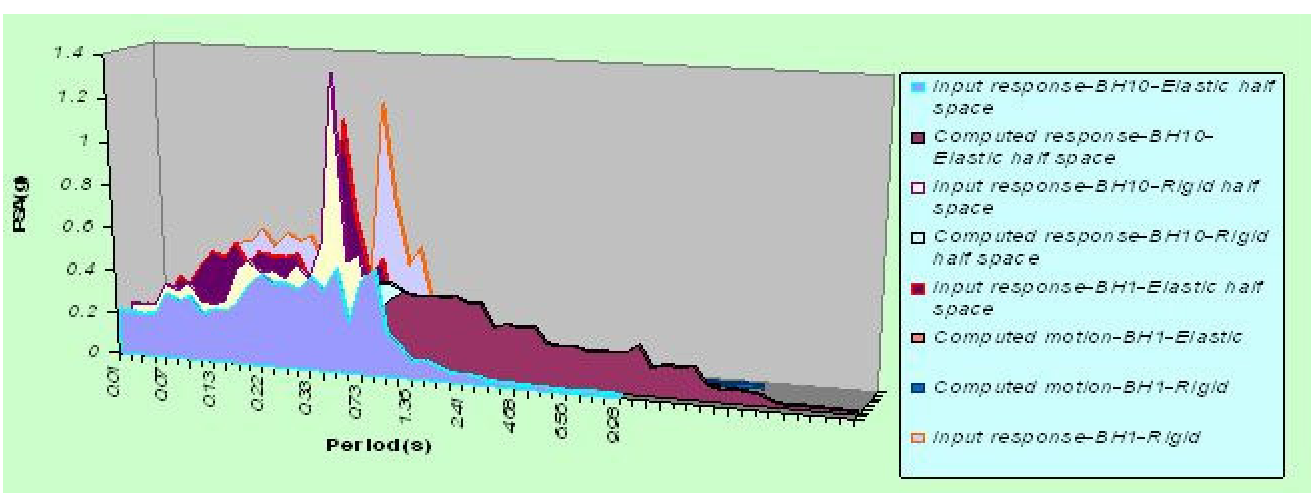

Figure 15. Comparison between input and computed response in different conditions by 5% damping ("Abbas Converter", Excel and Matlab).

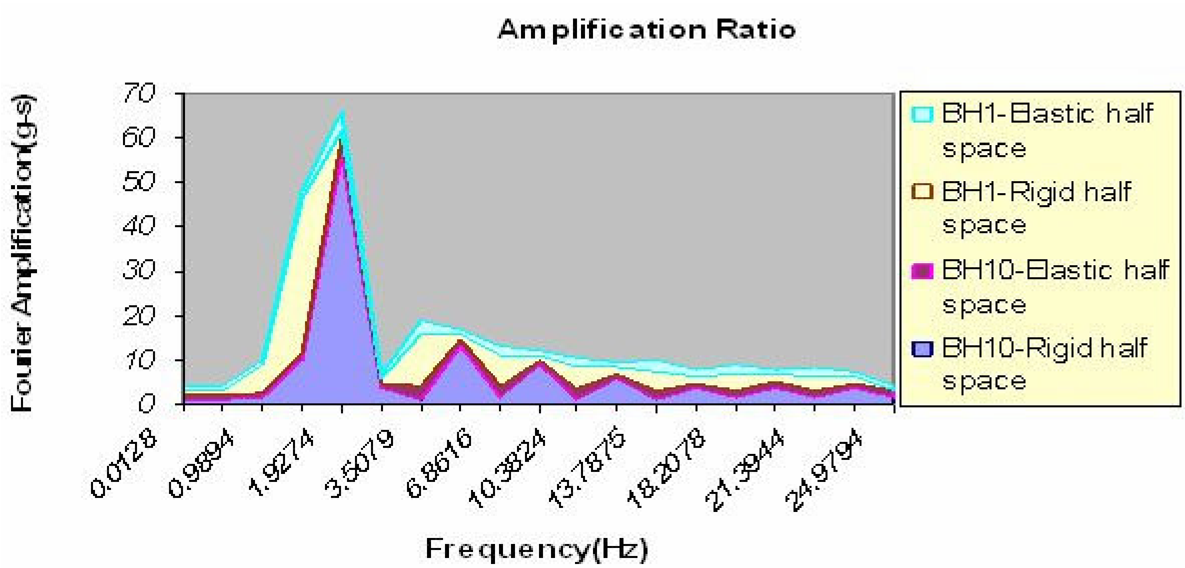

Figure 16. Amplification ratio spectrum.

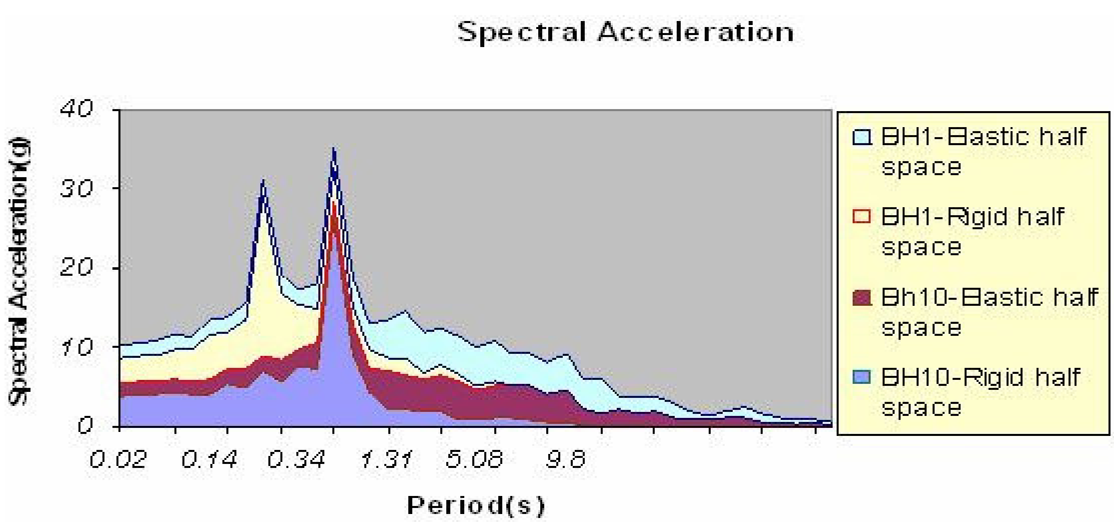

Figure 17. Spectral Acceleration spectrum ("Abbas Converter", Excel and Matlab).

ground motions for earthquake resistant design of structures. Many earthquakes in the past have left many lessons to be learnt, which are very essential to plan the infrastructure and even to mitigate such calamities in the future. Determination of the site specific ground response analysis is the aim of this effect of local soil conditions on seismic waves' amplification and hence, estimating the ground response spectra for future design purposes. The amplification spectrum of the soil column is computed between the top and the bottom of this soil deposit. Borings and dynamic *in situ* tests with the aim to evaluate the soil profile of V_s have been performed and the results show a very detailed and stable V_s profile. The obtained V_s profile has a good comparison with other *in situ* tests.

After evaluating the accelerograms at the bedrock, the ground response analysis at the surface, in terms of time history and response spectra, has been obtained by nonlinear standard hyperbolic model. The PGA value at the ground surface obtained from the used computer codes, which ranged from 1.1 to 0.57 g can be used to prepare the PGA map of Miyaneh. They are not distributed uniformly due to variation in the soil profile at various locations. More so, this PGA is comparable to the obtained peak horizontal acceleration values using SPT data, and the shape of the variation of peak acceleration with depth is similar to the SPT data. The calculated amplification factor ranged from 3.56 to 4.30 in elastic state, while 34.9 to 56.4 in rigid condition can be used to

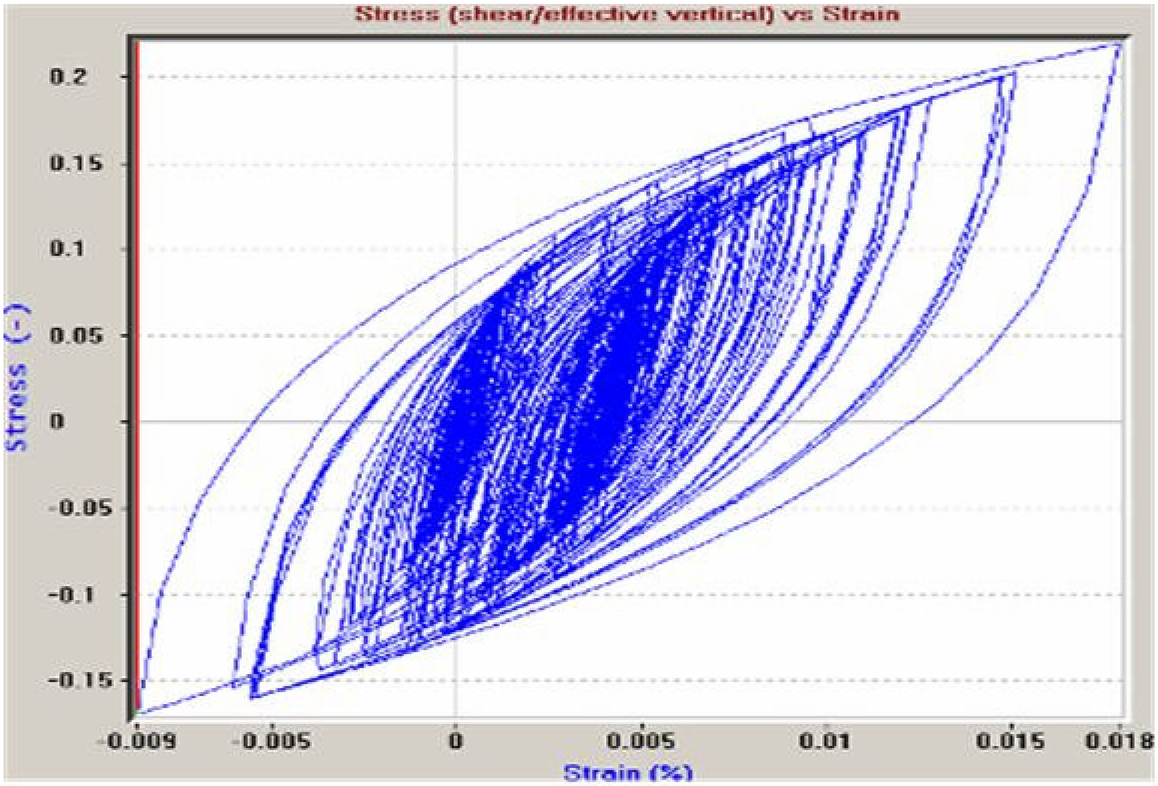

Figure 18. Elastic half space (BH10).

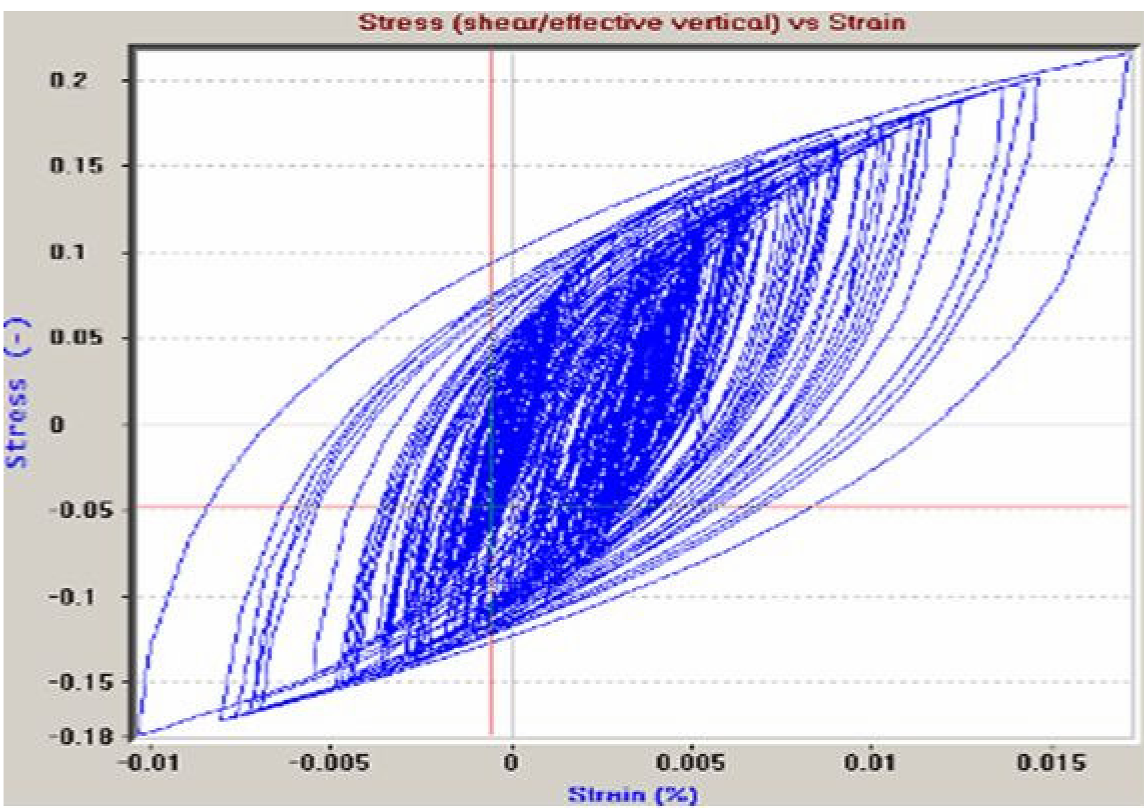

Figure 19. Rigid half space (BH10) (“Abbas converter” and UIUC).

Table 5. Rigid half space parameters.

Location	Parameter	Maximum at… (input)	Maximum at… (computed)
BH-10	Motion	0.2153 g (t = 24.1s)	0.0635 g (t = 39.1s)
BH-10	Stress	0.1581 (t = 24s)	0.2156 (t = 24.3s)
BH-10	Strain	0.1914% (t = 24s)	0.0171% (t = 24.3s)
BH-10	Fourier amplitude	0.575 (f = 1.86Hz)	0.2878 (f = 1.66 Hz)
BH-10	Fourier amplitude ratio	131.8 (f = 15.66Hz)	77.3 (f = 12.54Hz)
BH-10	Response spectra	PSA = 1.345 (period 0.54 s)	PSA = 0.3986 g (period 0.61s)
BH-1	Motion	0.2153 g (t = 24.1s)	0.0502 (t = 37.08 s)
BH-1	Response spectra	PSA = 1.171 g (period 0.54 s)	PSA=0.2963 g (period 0.56 s)
BH-10	Amplification ratio	---------------------------	56.4 (f =1.9274 Hz)
BH-1	Amplification ratio	---------------------------	34.9 (f =1.9146 Hz)
BH-10	Spectral acceleration	---------------------------	26.1 g (period 0.52 s)
BH-1	Spectral acceleration	---------------------------	8.22 g (period 0.65 s)

Table 6. Elastic half space parameters.

Location	Parameter	Maximum at… (input)	Maximum at… (computed)
BH-10	Motion	0.2027 g (t = 24.1 s)	0.0619 g (t =39.1 s)
BH-10	Stress	------------------------	0.219 (t = 24.3 s)
BH-10	Strain	------------------------	0.01789% (t = 24.3 s)
BH-10	Fourier amplitude	0.518 (f = 1.86 Hz)	------------------------
BH-10	Fourier amplitude ratio	121.5 (f = 15.66 Hz)	-----------------------
BH-10	Response spectra	PSA =1.237 g (period 0.54 s)	PSA = 0.3842 g (period 0.55s)
BH-1	Motion	0.2027 g (t = 24.1 s)	0.0619 g (t =39.1s)
BH-1	Response spectra	PSA = 1.112 (period 0.54 s)	PSA=0.2936 g (period 0.55 s)
BH-10	Amplification ratio	---------------------------	3.56 (f =1.8889 Hz)
BH-1	Amplification ratio	---------------------------	4.30 (f =1.8889 Hz)
BH-10	Spectral acceleration	---------------------------	4.91 g (period 0.53 s)
BH-1	Spectral acceleration	---------------------------	5.79 g (period 0.52 s)

prepare the amplification map of Miyaneh region.

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