A proposed procedure for nonlinear site response evaluation on strong ground motion during Ardabil earthquake (28 Feb. 1997), by using "Abbas Converter" computer code

Abbas Abbaszadeh Shahri [1], Dr.Bijan Esfandiari [2] and Dr.Katayoun Behzadafshar [3]

Abstract

In this paper a case study on ground response analysis of a site in Miyaneh region during the Ardabil earthquake (28 Feb. 1997, M_w 6.1) is presented. The Miyaneh city and its suburban areas are located in the Northwest of Iran in Eastern Azarbayjan province. This area is prone to high seismic risk due to the presence of several active faults. Subsurface soils subjected to strong motion exhibit significant nonlinear behavior. For site characterization, deep site investigations have been undertaken, and a seismic geotechnical procedure for the proposed bridge over the rivers at 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 assumption of elastic and rigid (viscoelastic) half space bedrock by using of standard hyperbolic model nonlinear approach was evaluated and the results of them were compared to each other. Test of the capability of designed computer code by authors, namely as "Abbas Converter", description and evaluating 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 studied area 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 is worked and installed so quickly, operated as a logic connecter function between the used softwares. Therefore, it can make and render the study easier than previous have done, and take over the encountered problems.

Keywords: nonlinear site response, strong ground motion, Iran railway net work.

Introduction[123]

The area of Iran is 1648195 Km^2 with more than 70 millions population which is located in south west of Asia in the Middle East. Because of its special strategic location, during the past centuries, has operated as connecter between the gap of West and East 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. In the second half of the 19th century, during the time of the third king of Ghajar chain, a short horse-driven suburban railway was established in south of Tehran that was later converted to steam. This line was closed in 1952. 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

[1] Member of faculty, Islamic Azad University, Hamedan Branch, email: a_abbaszadeh@iauh.ac.ir,
[2] Member of faculty, Engineering faculty of Tehran University
[3] Member of faculty, Islamic Azad University, Shahre-Rey unit, email: katayoun.afshar@gmail.com

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 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 unknown. Several stretches of line, including tunnels, were built through unsuitable geology, 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 vast majority of the engines were diesel-powered. The majority of transportation in Iran is road-based. 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.

The Miyaneh region with 47˚, 30΄ to 48˚ East longitude and 37˚ to 37˚, 30΄ North latitude is located on Northwest of Iran in Eastern Azarbayjan province. On base 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, has a rough topography with high deep and V shape valleys. Nearly total of region outcrops are Cenozoic volcanic and sedimentary rocks. Only in a small portion of Ghafelankuh Mountains because of faulting, Permian-Terries carbonate sediments have exposures. This area is an active seismic belt which is located in Alborz-Azarbayjan seismotectonic province. Regarding National Geodatabase of Iran website (*www.ngdir.ir*) the earthquake events (historical events are ignored) with magnitudes more than 4.5 are given in table (1).

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

Date	M_b	M_s	Long.	Lat.	Reference	Event time
002/04/07	---	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
993/03/15	4.7	4	45.826	38.125	ISC	15:32:38
989/12/03	4.8	4	45.351	38.442	ISC	07:39:11
989/12/02	4.5	---	45.425	38.453	ISC	04:51:59
1988/01/07	4.8	---	45.528	38.478	ISC	10:59:39
986/07/10	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
980/10/10	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
965/02/10	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
931/07/04	---	5	45.6	38	KAR	21:00:50
931/05/12	---	5	46.3	38.8	KAR	10:25:13
930/05/29	---	6	45.5	37.5	KAR	17:14:55
930/05/23	---	5	45.5	37.5	KAR	09:48:20
930/05/08	---	5	45.5	37.5	KAR	15:05:21
928/03/24	---	5	47.3	37.8	KAR	10:53:16

The Ardabil earthquake with M_w 6.1, depth of 10 Km and 38.075 N, 48.050 E epicentral coordination was a destructive earthquake that occurred on 28 Feb. 1997. The epicenter was located near the city of Ardabil in northeastern Iran. This event occurred 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)* At least 1100 people were killed, 2600 injured, 36000 homeless, 12000 houses damaged or destroyed and 160000 livestock killed in this area of Iran. Severe damage was observed to roads, electrical power lines, communications and water distribution systems around Ardabil (Person and Waverley 2008). 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 use of geological, geophysical and geotechnical data with some software such as Seismosignal, LisCADv6.2, Log2.1, Proshake, SMSIM , Curve expert 1.3, UIUC developed software, MATLAB programming environment and a designed computer code by authors namely as "Abbas converter" the response spectra, computed motion and some related parameters for the selected area were evaluated and compared. Because of the limitation in software applicability, no one of above softwares can reply to all requested parameters lonely. As known, any of them use specific format and their data aren't applicable for each other and our study respectively. For this reason the authors forced to produce a computer program to generate the new motions for them and convert the primary input data of mentioned softwares to each other. This is the main reason for designing the "Abbas Converter". This produced code has several which will describe in analysis method section. If the recorded time history for selected region was not available, it would be forced to use artificial motion (time history) to predict the surface response spectra and this is the reason for using the SMSIM code.

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 of the importance of site effect on ground motions. This problem is commonly referred to site specific response analysis or soil amplification study. Therefore accounting into such effects has gained critical importance in seismic regulations, land use planning and seismic design of critical facilities, to obtain 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. Some of the soil conditions and local geological features affecting the ground response are as below:

- Horizontal extent and depth of the soil deposits overlying bedrock.
- Slope of the bedding planes of the soils overlying bedrock.
- Horizontal changes in soil types.
- Topography and geometry of both bedrock and deposited soils.

The effect of local geology on ground motion propagation is significant and can not be ignored. 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 available response analysis methods needs to practice and increase knowledge about the basic soil behavior under cyclic loading which 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 namely equivalent linear analysis method (frequency domain solution) and nonlinear analysis method (time domain solution).

Yoshida (1994), Huang et al. (2001) and Yoshida and Iai (1998) showed that equivalent linear analysis shows larger peak acceleration because the method computes and takes into account the acceleration in high frequency range large. 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 strain vibration during loading is significant and can not be approximated by representative strain throughout the duration of shaking, thus 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 determination of the seismic ground motion at a given site, due to an earthquake incite, with a given intensity and epicentral distance. Use a wide database of recorded strong motions and to group accelerograms with similar source, path and site effects could be the ideal solution for such a problem, which in practice such a database is not available. An alternative way for taking over to this problem is based on computer codes, developed from the knowledge of the seismic source process and of the propagation of seismic waves, that can simulate the ground motion associated with the given earthquake scenario. In such a way, synthetic signals, to be used as seismic input in a subsequent engineering analysis, can be produced at a very low cost/benefit ratio (Borja et al. 1999, Elgamal et al. 1996). The objective of a site response analysis is to estimate the ground shaking during an earthquake, which is shaking at sites that does not include effects caused by proximity to structures or topographic features, for a specific hazard level and set of site conditions. The requisite components for a site response analysis are: one or more design earthquake events with representative earthquake record(s), an idealization of the soil-rock system at the site of interest, and a scheme to generate response solutions to simplified assumed wave fields. Normally, the ground response is presented in terms of either response spectra or the variation of acceleration or velocity with time. Traditionally, earthquake ground motions are predicted in two stages.

1) Using an attenuation relationship to relate the earthquake magnitude

2) Using a response spectrum model to define the design response spectrum (Boominathan 2004, Lam et al. 2000).

Site effect, response and amplification

The importance of site effect on seismic motion has been realized since 1920s and quantitative studies have been conducted using strong motion array data after 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 the local soils acts as a filter and modifies the ground motion characteristics, is known as 'soil amplification' (Anbazhagan 2007*)*. The term amplification factor is used here to refer to 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 demonstrated by past earthquakes. While there are potentially other factors contributing to damage (topographic and basin effects, liquefaction, ground failure or structural deficiencies), the amplification of ground motion due to local site conditions plays an important part in increasing seismic damage (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. Site effect and responses are associated with:

1. Superficial deposits: Earthquake ground motion can be significantly amplified by superficial deposits. Even though seismic waves generally travel tens of kilometers of rock and less than 100m of soil, the soil plays a very important role in determining the characteristics of ground motion (Kramer 1996), therefore understanding of site response of geological materials under seismic loading is an important element in developing a well-established constitutive model.

2. Topographic and basin effects, Liquefaction, Ground failure and Structural deficiencies: these 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).*

3. Profile depth: Site response is also a function of profile depth, thus ignoring profile depth may have a detrimental effect in ground motion prediction and have also been introduced into most current attenuation relationships. However, most attenuation relationships account for site effects only through a broad site classification system that divides sites into "rock and shallow stiff soil" , "deep stiff soil" ,and "soft soil" (Park and Hashash 2004; Rodriguez et al. 2000).

4. 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.

5. Soil type: The effect of nonlinearity is largely a function of soil type (Vucetic 1990; Vucetic and Dobroy 1991, Sitharam et al. 2004).

6. 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 measuring the dynamic stiffness of the site and depth of the deposit (Rodriguez et al. 2000).

7. Frequency of the base motion, the geometry and material properties of the soil layer above the bedrock: The response of a soil deposit is dependent on these factors.

To account partially for these factors, a site classification scheme should include the nonlinear behavior of soil and measuring 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 ground under earthquake loading and excitation. Some of them have described at the present paper as below:

1- Seed and Idriss (1970), Joyner and Chen (1975): investigated the effects of site parameters such as secant shear modulus, depth of bed rock, types of sand and clay (these three parameters have a significant effect on results of site response analysis), low-strain damping ratio and location of water table (these 2 parameters have a minor effect on results of site response analysis).

2- Schnabel et al. (1972): proposed Equivalent linear approach that is widely used for site response analysis.

3- Seed et al. (1976): based on the statistical study of 147 recordings from western U.S earthquakes of about magnitude 6.5, developed peak acceleration attenuation relation ships for different site conditions.

4- Idriss (1990): developed an empirical correlation between the peak acceleration at rock outcrop and soft soil. The relation is based on recordings from Mexico City (1985) and Loma Prieta (1989).

5- Kramer (1996): developed a nonlinear approach as by this method a nonlinear inelastic stress-strain relationship is followed in a set of small incrementally linear steps.

6- Field et al. (1997): the view of geotechnical engineers, based largely on laboratory studies, is Hook's law (linear elasticity) breaks down at larger strains causing a reduced (nonlinear) amplification.

7- Borja et al. (1999): developed a fully 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 20 may 1986earthquake.

8- Rodriguez et al. (2001): proposed an empirical geotechnical seismic site response procedure that accounts the nonlinear stress-strain response of earth materials under earthquake loading.

Linear and nonlinear behavior

Soil behavior is nonlinear when shear strains exceed about 10^{-5} (Hardin and Drenvich 1972). The nonlinear behavior of soils is the most important factor in ground motion propagation and should be accounted 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 tangent shear modulus (G_{tan}). The secant shear modulus (G_{sec}) is the average shear modulus for a given load cycle. The G_{sec} decreases with increase in cyclic shear strain. Instead of defining the actual hystersis loop, the cyclic soil behavior is often represented as shear modulus degradation and damping ratio curves. The shear modulus degradation curve relates secant shear modulus to cyclic shear strain, whereby shear modulus is normalized by the maximum or initial shear modulus.

Insitu 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).

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

$u_s(z_s,t) = A_s e^{i(\omega t + k_s^* z_s)} + B_s e^{i(\omega t - k_s^* z_s)}$ (1)

$u_r(z_s,t) = A_r e^{i(\omega t + k_r'' z_s)} + B_s e^{i(\omega t - k_r'' z_s)}$ (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

$\tau(0, t) = G_s^* \gamma(0, t) = G_s^* (\partial u_s(0,t)/ \partial z_s) = 0$ (3)

Where $G_s^* = G(1+2i\xi)$ 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:

$u_r(z_j, t) = (A_j e^{ik_j^* z_j} + B_j e^{-ik_j^* z_j}) e^{i \omega t}$ (4)

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

$A_{j+1} + B_{j+1} = A_j e^{ik_j'' h_j} + B_j e^{-ik_j'' h_j}$ (5)

Continuity of shear stresses requires that:

$A_{j+1} + B_{j+1} = G_j^* k_j^* / G_{j+1}^* k_{j+1}^* (A_j e^{ik_j'' h_j} - B_j e^{-ik_j'' h_j})$ (6)

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

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

$R_\gamma = (M-1)/10$ (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 (e.g. input motion imposed at the bottom of the soil column) using the transfer function that relates displacement amplitude at layer *i* to that the layer *j*:

$F_{ij}(\omega) = |u_i|/|u_j| = (A_i(\omega) + B_i(\omega))/ (A_j(\omega) + B_j(\omega))$ (9)

$|\ddot{u}| = \omega |\dot{u}| = \omega^2 |u|$ (10)

For harmonic motions and the transfer function can be used to compute accelerations and velocities. Main reason using linear approach is 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 analysis 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 1-D analysis of shear wave propagation in layered soil media. When compared with earthquake observation, nonlinear analysis 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 to model the stress-strain soil behavior of soils subjected to constant rate of loading. The hyperbolic equation is defined as:

$\tau = (G_{mo}\gamma)/1 + [(G_{mo}/\tau_{mo})\gamma] = (G_{mo}\gamma)/[1+(\gamma/\gamma_r)]$ (11)

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 strain at which failure would occur if soil were to behave elastically. It has been considered a material constant by Hardin and Drenvich *(1972)*. The reference strain can also be represented as function of initial tangent modulus and undrained shear strength in clays (Mersi et al. *1981)*. The hyperbolic model has been implemented in many site response analysis codes, such as DERSA.

One of the most reliable methods to characterize $G_{max,}$ is insitu measurement of V_s in the field at small strain using seismic methods (Rolling et al. 1998*)*. On the ground surface at strain levels less than 0.001%, G_{max} can be determined from the measured V_s profile by assuming the density (ρ) as:

$G_{max} = \rho V_s^2$ (12)

G_{max} can also be estimated directly from N values in the field as:

$G_{max} = a N^b$ (a, b: correlation coefficients) (13)

Several correlations are reported between V_s and N values measured in the field and are comprehensively summarized in table (1), which are often expressed in the following form:

$V_s = AN^B$ (14)

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 (,Imai *1977*, Imai and Tonouchi *1982*). Estimation of V_s can be improved, if the effective stress is included in the regression equation. Similarly, table (2) can be used to estimate G_{max} by assuming the density of soil, since slight variation of density does not influence the estimated value.

Table (2): V_s- N correlation reported in papers (Hanumantharao and Ramana 2008)

Author(s)	Correlation	Soil	Country
Imai & Yoshimura,1970	V_s=76.0N$^{0.39}$	All	Japan
Ohba & Toriumi,1970	V_s=84.0N$^{0.31}$	Alluvial	Japan
Shibata, 1970	V_s=32.0N$^{0.50}$	Sands	Japan
Ohta et al., 1972	V_s=87.0N$^{0.36}$	Sands	Japan
Ohsaki & Kawasaki, 1973	V_s=82.0N$^{0.39}$	All	Japan
Ohsaki & Kawasaki, 1973	V_s=59.0N$^{0.47}$	Cohesionless	Japan
Imai et al., 1975	V_s=90.0N$^{0.34}$	All	Japan
Imai, 1977	V_s=91.0N$^{0.34}$	All	Japan
Ohta & Goto, 1978	V_s=85.0N$^{0.35}$	All	Japan
JRA, 1980	V_s=100.0N$^{0.33}$	Clays	Japan
JRA, 1980	V_s=80.0N$^{0.33}$	Sands	Japan
Imai & Tonouchi, 1982	V_s=97.0N$^{0.31}$	All	Japan
Yokota et al., 1991	V_s=121.0N$^{0.27}$	All	Japan
Seed & Idriss, 1981	V_s=61.0N$^{0.50}$	All	USA
Seed et al., 1983	V_s=56.4N$^{0.50}$	Sands	USA
Sykora & Stokoe, 1983	V_s=106.7N$^{0.27}$	Granular	USA
Fumal & Tinsley, 1985	V_s=152+5.1N$^{0.27}$	Sands & ravelly sands	USA
Sykora & Koester, 1988	V_s=63.0N$^{0.43}$	Holocene gravels	USA
Sykora & Koester, 1988	V_s=132.0N$^{0.32}$	Pleistocene gravel	USA
Lee, 1990	V_s=57.0N$^{0.49}$	Sands	USA
Lee, 1990	V_s=114.0N$^{0.31}$	Clays	USA
Lee, 1990	V_s=106.0N$^{0.32}$	Silts	USA
Rollins et al., 1998	V_s=63.0N$^{0.43}$	Holocene gravels	USA
Rollins et al., 1998	V_s=132.0N$^{0.32}$	Pleistocene gravel	USA
Rollins et al., 1998	V_s=222.0N$^{0.06}$	Recent fill	USA
Andrus rt al., 2004	V_s=87.8N$^{0.25}$	All	USA
Pitikilas et al., 1992	V_s=155.1N$^{0.17}$	Debris fill	Greece
Pitikilas et al., 1992	V_s=162.0N$^{0.17}$	Silty sand	Greece
Pitikilas et al., 1992	V_s=165.7N$^{0.19}$	Soft clay	Greece
Pitikilas et al., 1992	V_s=357.5N$^{0.19}$	Hard clay	Greece
Kalteziotis et al., 1992	V_s=76.2N$^{0.24}$	All	Greece
Kalteziotis et al., 1992	V_s=76.6N$^{0.45}$	Cohesive soil	Greece
Kalteziotis et al., 1992	V_s=49.1N$^{0.50}$	Cohesionless soil	Greece
Athanasopoulos, 1995	V_s=107.6N$^{0.36}$	All	Greece
Raptakis et al., 1995	V_s=123.4N$^{0.29}$	Loose	Greece
Raptakis et al., 1995	V_s=100.0N$^{0.24}$	Medium dense sand	Greece
Raptakis et al., 1995	V_s=105.7N$^{0.33}$	Soft clay	Greece
Raptakis et al., 1995	V_s=184.2N$^{0.17}$	Stiff clay	Greece

Raptakis et al., 1995	V_s=192.4$N^{0.13}$	Gravel	Greece
Jafari et al., 1997	V_s=22.0$N^{0.85}$	All	Iran
Jafari et al., 2002	V_s=27.0$N^{0.73}$	Clays	Iran
Jafari et al., 2002	V_s=22.0$N^{0.77}$	Silts	Iran
Jafari et al., 2002	V_s=19.0$N^{0.85}$	Fine grained soil	Iran
Chein et al., 2000	V_s=22.0$N^{0.76}$	Silty sand	Taiwan
Kayabali, 1996	V_s=175+3.75N	Granular	Turkey

Analysis method

Analysis method steps are as follows:

- Characterization of site based on field investigation and laboratory test.
- Elect and apply the rock motion (natural or synthetic acceleration time histories) on soil profile column for rigid and elastic half space bedrock associated with seismotectonic structure to represent the effect of motion for the site on the soil profile. Using the rock time history as input motion, ground response analysis is conducted for the modeled soil profiles to compute ground motion at the surface. Response spectra of the motions of the surface are computed for various analysis made.
- Analysis of site response, develop and improvement of site surface response spectra.

Figure 1 shows the general steps of analysis.

The input motion as rock motion at bedrock was performed by Seismosignal. The comparison of modulus reduction curves for selected boreholes, corresponding to defined input motion, V_s profile and finding G_{max} after soil definition was done by Proshake. If the natural recorded time history for selected region was not available, it would be forced to use SMSIM code to generate an artificial motion or time history to evaluate the surface response spectra. Regression models (both linear and nonlinear) for various interpolation schemes were performed and compared by Curve Expert1.3 and MATLAB programming environment. To obtain the response of site in nonlinear state the UIUC was perform and executed. Due to limitation in software applicability, no one of mentioned softwares can reply to all requested parameters lonely. Therefore the authors forced to produce a computer program “Abbas Converter”to solve this problem. This produced code has several advantages such as:

1. Work and installs so quickly.

2. Operates as a logic connecter link between the used softwares.

3. Can generate the input data correspond the defined format for the used softwares.

4. Its output results can easily export to the other used software in this study.

5. This designed software make and render easy the study more than previous have done.

6. With it, the authors could enter recorded data with different format as an input and take defined format for the used softwares.

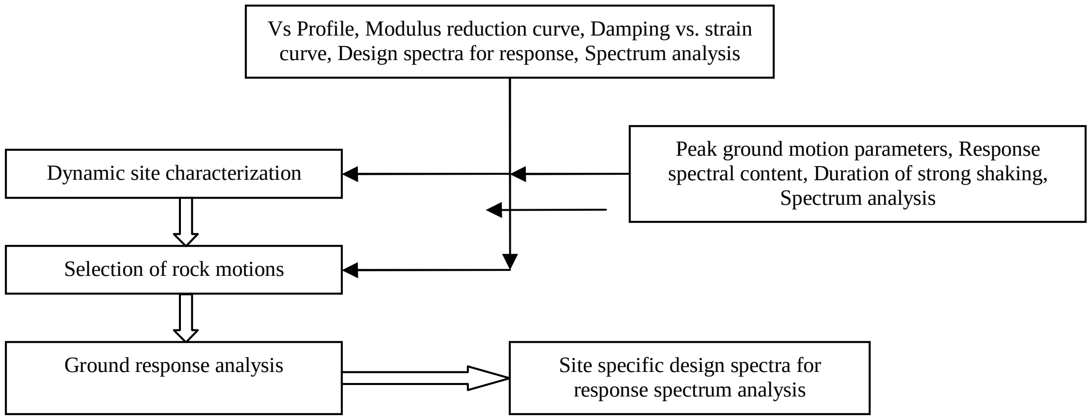


Figure1. Site specific ground response analysis.

As illustrated above, the proposed steps of this study are suggested as shown in Figure2 and 3. This procedure indicated that the designed computer code can work in to different conditions and shows that ”Abbas Converter” facilitate and render easy the procedure more than before. With this method authors could enter new and different format of data as an input and take defined format for the software. For soil properties modeling in nonlinear time domain analysis in this study, the standard hyperbolic model with elastic half space bedrock for first analysis and rigid half space for the second one, flexible time control, maximum strain increment about 0.005 and damping matrix defined with modes and frequency were selected. In addition, it is important to note that, the accuracy of time domain solution depends on the time steps. With these codes, it is possible to:

- Generate the animation of column displacement subjected to selected input motion.
- View the changing of the soil column at each step of input motion duration time.

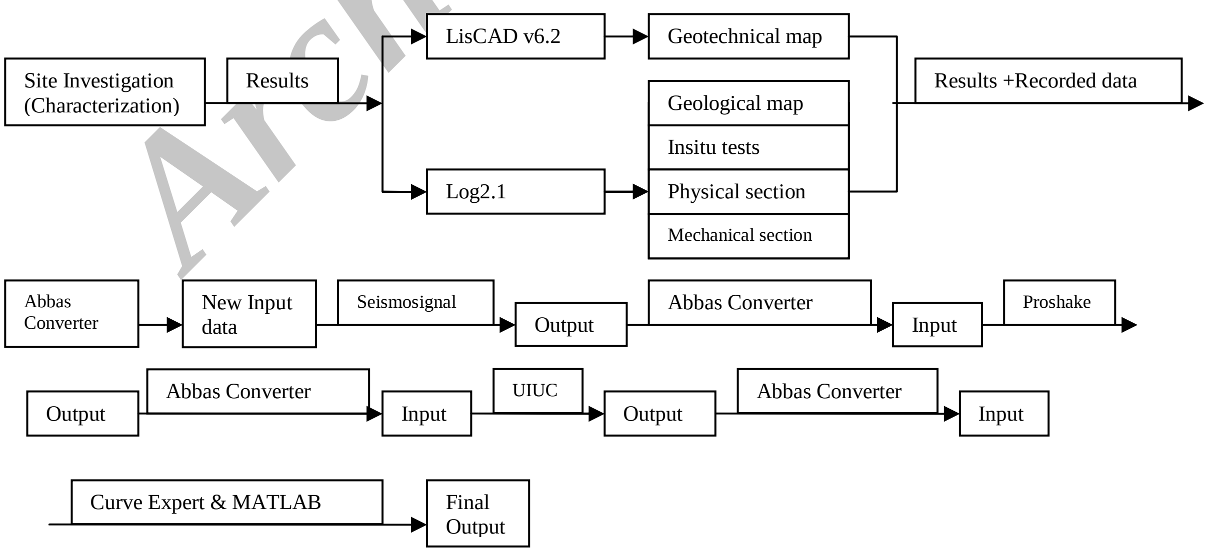


Figure2. Proposed method for this study by authors.

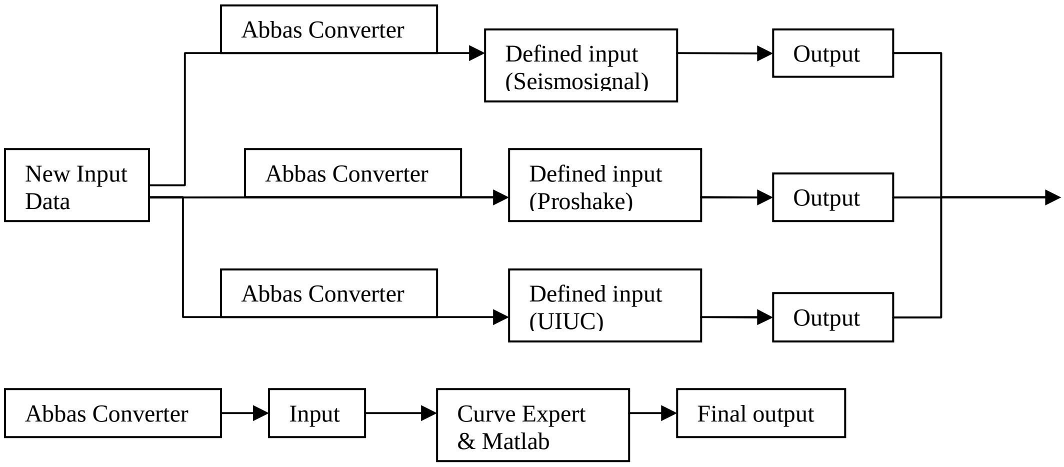


Figure3. Ability of "Abbas converter" to install in parallel condition.

This study was done by use of recorded data of Ardabil event (1997 M_w 6.1) in Ardabil province of Iran. The L component of Ardabil event at bedrock was applied to each borehole location based the hypocentral distance calculated for each of them in the Miyaneh region to study the site response as shown in Figure 4, 5and 6. The recorded data was picked up from BHRC web site of Iran and by use of "Abbas Converter" were drawn with seismosignal.

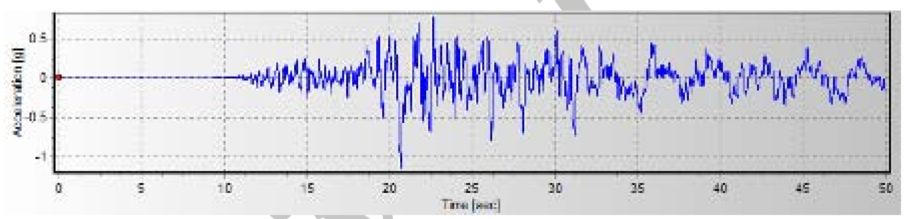


Figure4. L Component of Ardabil event (PGA=1.1447g at t=20.66s).

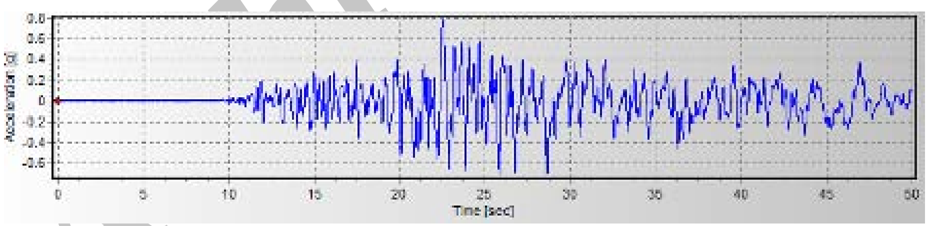


Figure5. V Component of Ardabil event (PGA=0.584412g at t=18.94s).

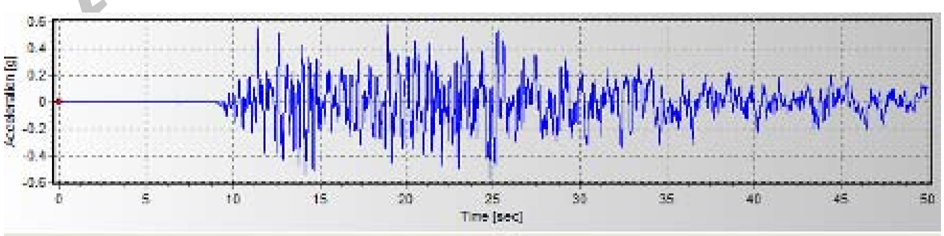


Figure6. T Component of Ardabil event (PGA=0.778382g at t=22.5s).

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 minimum 40m depth namely as BH1and BH10 were select and presented. Figure7 shows the operational steps and testing program.

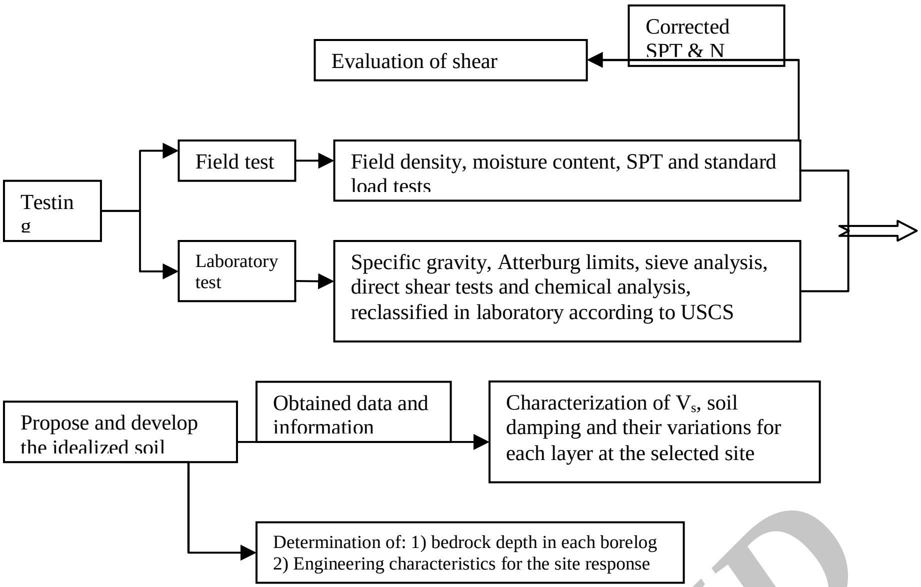


Figure7. Testing program steps

In these siteswater table was find up to depth of 9m for BH10 and 5m for BH1 at the time of drilling operation. Soil profile as shown in Table (3) and (4), for comparison must be created and modified. V_s of surficial sediments were investigated in correlation with geotechnical properties determined by laboratory testing and in addition lithofacies based on detailed core investigation were taken in to account of the correlation analysis. N values (one of the most common parameters for the evaluation of geotechnical properties of soils) obtained by insitu 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. Despite of its incorrectness, N value is quite attractive because of its existence of large amount of data at 1m interval, which makes it easy to correlate with V_s. 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 (5).

Table (3): 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=1016.125m/s						

Table (4): 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 $\gamma(2.21gr/cm^3)$, Vs=1214.2m/s						

Table (5): Correlation results of Vs- N for selected region by curve expert1.3 and MATLAB

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

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

From the wave propagation theory, it is clear that the ground motion amplitude depends on the density and V_s of subsurface material. Usually insitu density has relatively smaller variation with depth and thus the V_s is the logical choice for representing site conditions. By obtained information from drilled holes the shear modulus and V_s and their variations for each soil layer will present at the investigated site as shown in figure 8, 9, 10 and 11.

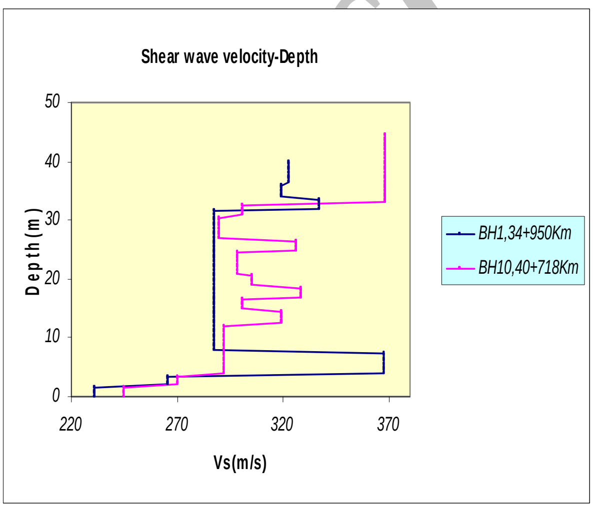


Figure8.Vs-Depth profile.

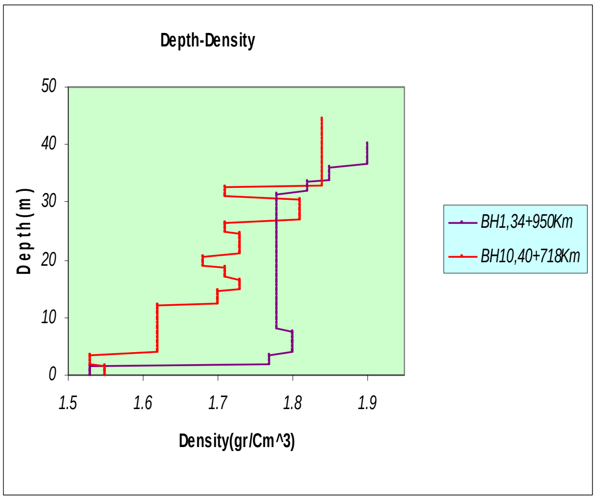
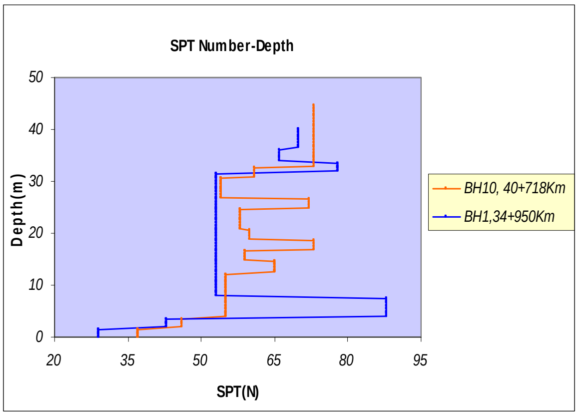


Figure9. Density-Depth profile.



Figurte10. SPT number-Depth profile.

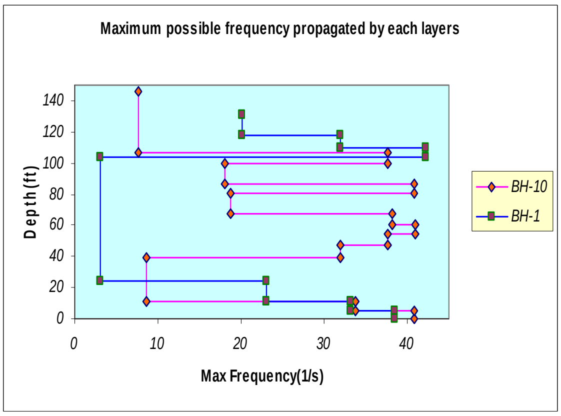


Figure11. Maximum frequency by each layer.

The time histories as input motions are applied on the bottom of profile and assigned to measure at 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 12. Site response analysis is conducted for modeled idealized, develop 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 Figure13. Regarding the knowledge of motions is based on recording at rock outcrops and unless the rock is rigid, the motions at he base of the soil profile will differ from those of outcrop. Furthermore amplification spectrum between the first and top layer can tbe obtained.

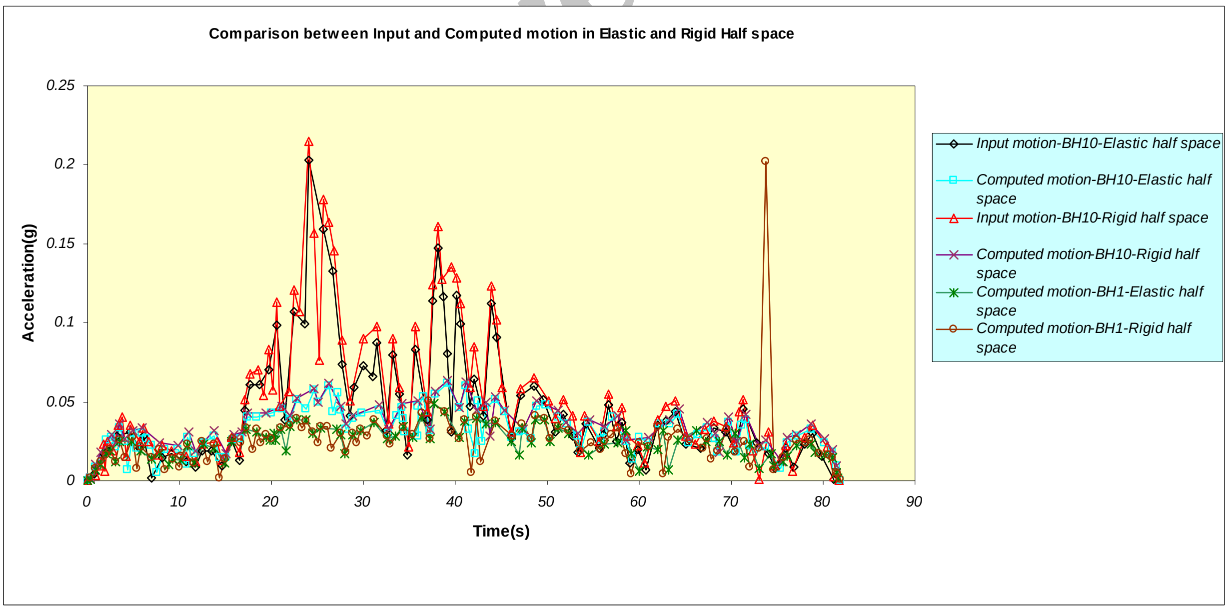


Figure12. Comparison between the input and computed motion in elastic and rigid half space (5% damping).

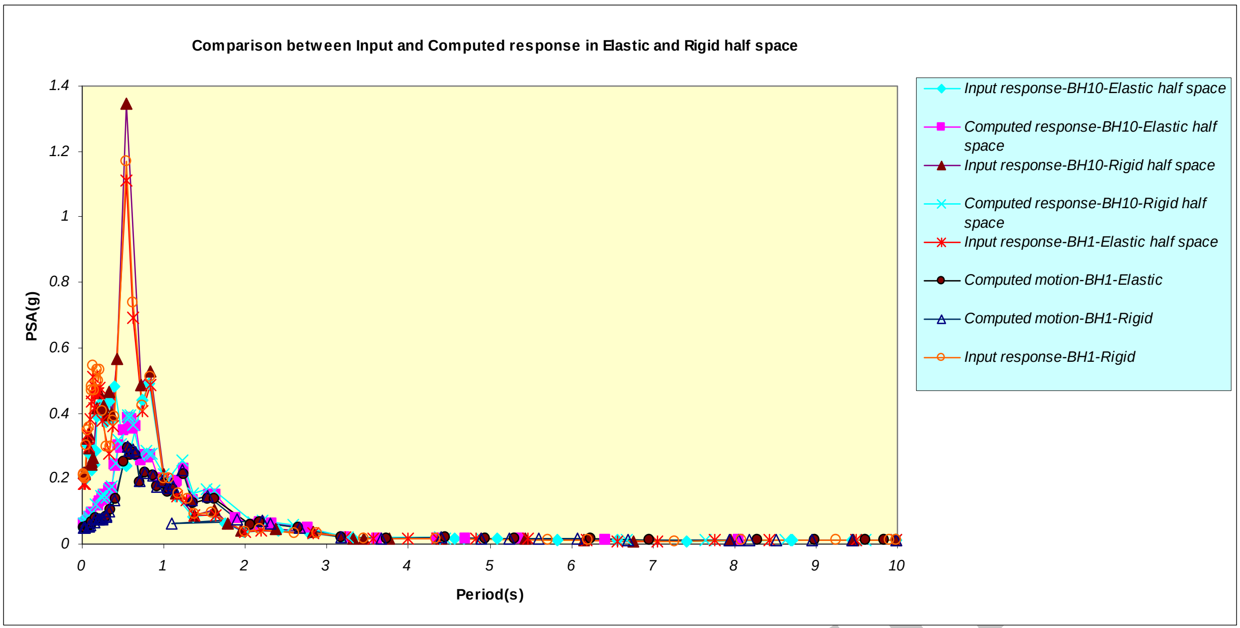


Figure13. Comparison between input and computed response in elastic and rigid half space (5% damping).

The term amplification factor is used here to refer to the ratio of the peak horizontal acceleration at the ground surface to the peak at the bedrock which is evaluated by using the obtained PGA at bedrock from the applied acceleration time history for each borehole and the peak ground surface acceleration obtained as a result of ground response analysis. These results are given in Figure 14 and 15. Figure16 and 17 displays the Fourier amplitude spectrums for two boreholes in different conditions.

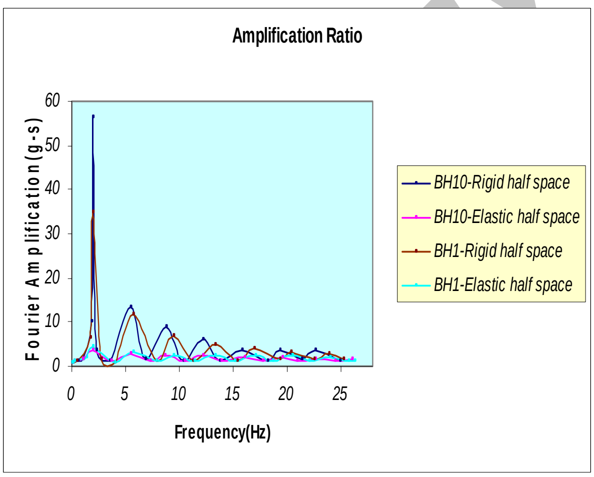


Figure14. Amplification ratio spectrum.

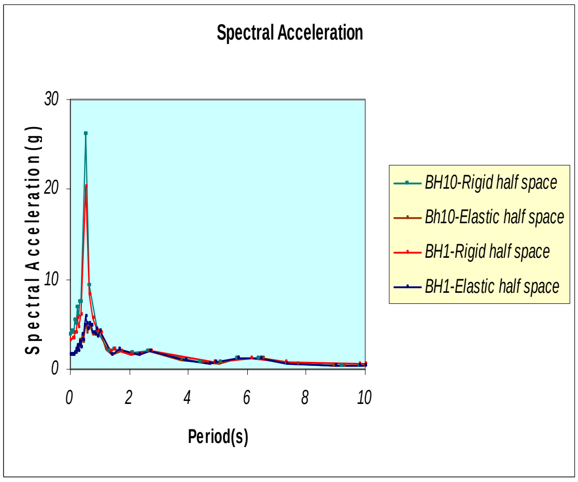


Figure15. Spectral Acceleration spectrum.

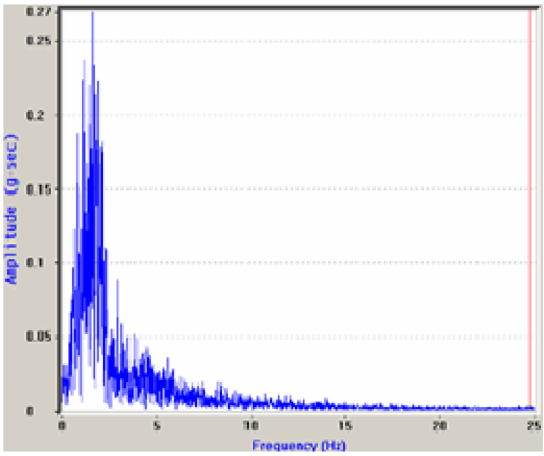


Figure16.Elastic half space (BH10).

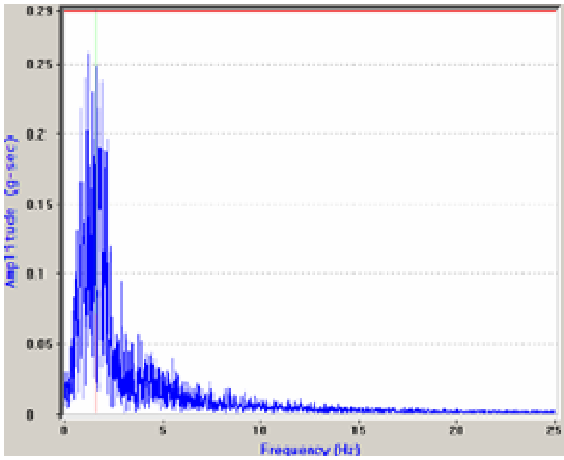


Figure17.Rigid half space (BH10).

Stress-strain time histories were presented in Figure18, 19 for elastic and rigid half space. PGA profiles in elastic and rigid bedrock were shown in Figure20, 21, 22 and 23. As given in Figure24, 25, 26 and 27 the permanent displacement was computed for the borehole locations for elastic and rigid half spaces.

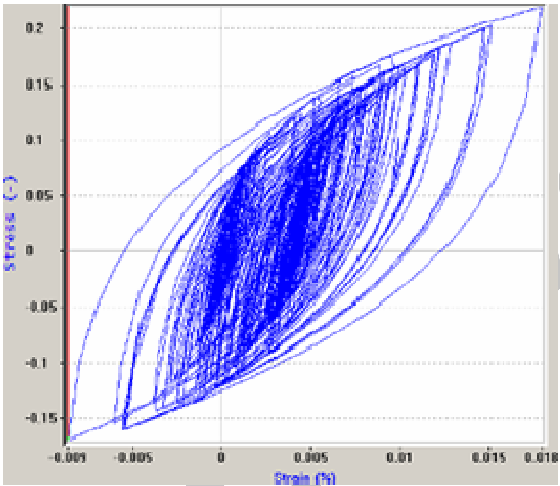


Figure18.Elastic half space (BH10).

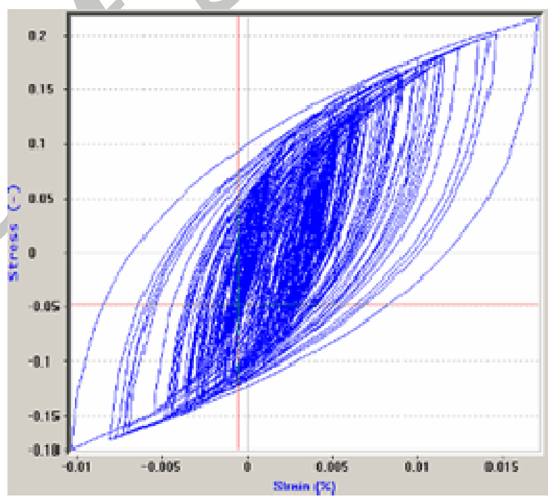


Figure19.Rigid half space (BH10).

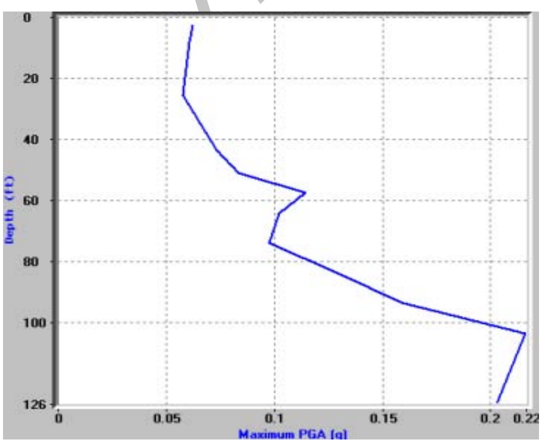


Figure20.Elastic half space (BH10).

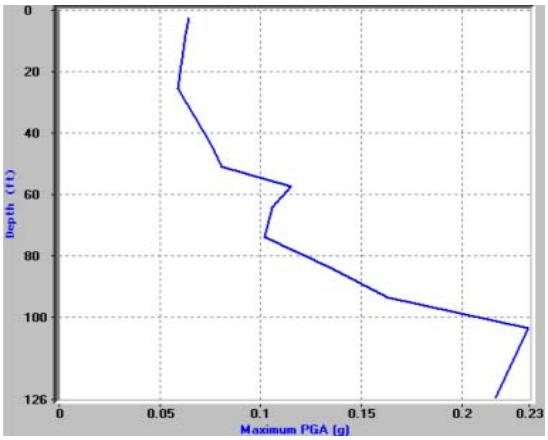


Figure21.Rigid half space (BH10).

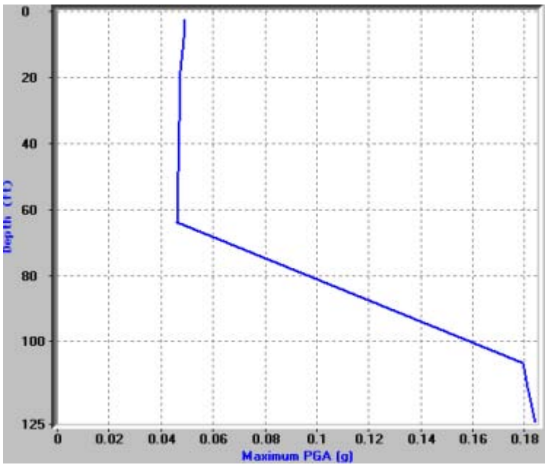


Figure22.Elastic half space (BH1).

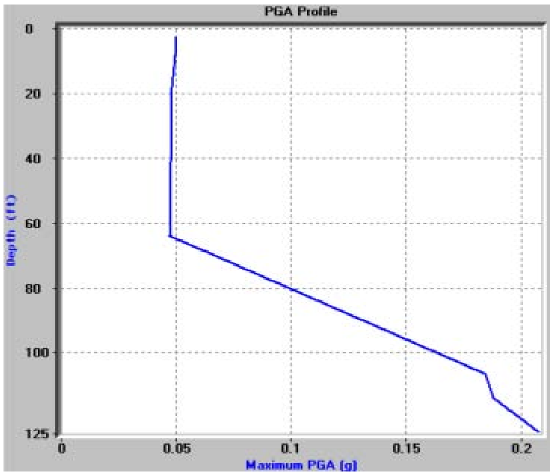


Figure23.Rigid half space (BH1).

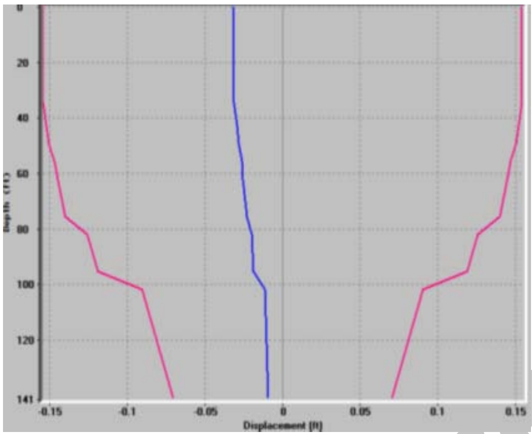


Figure24.Elastic half space (BH10).

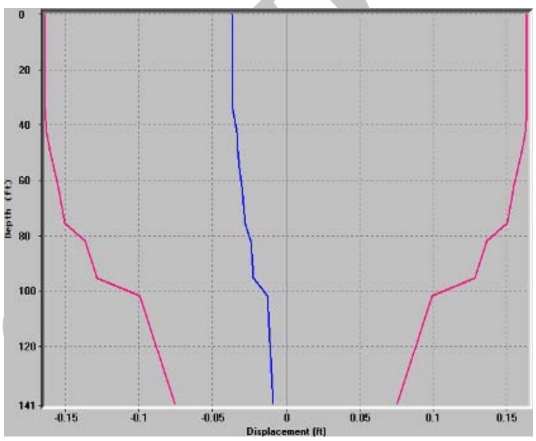


Figure25.Rigid half space (BH10).

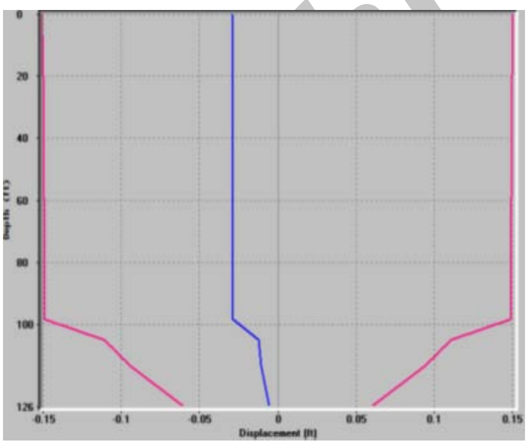


Figure26.Elastic half space (BH1).

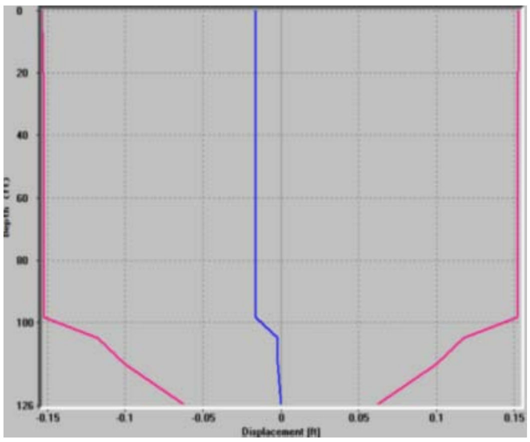


Figure27.Rigid half space (BH1).

By comparison of the above figures the following results can under take and summarize in table (6) and (7).

Table (6): Rigid half space parameters

Location	Parameter	Maximum at…(Input)	Maximum at…(Computed)
BH-10	motion	0.2153g (t=24.1s)	0.0635g (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=1.66Hz)
BH-10	Fourier Amplitude Ratio	131.8 (f=15.66Hz)	77.3 (f=12.54Hz)
BH-10	Response spectra	PSA=1.345 (Period 0.54s)	PSA=0.3986g (Period 0.61s)
BH-1	motion	0.2153g (t=24.1s)	0.0502 (t=37.08s)
BH-1	Response spectra	PSA=1.171g (Period 0.54s)	PSA=0.2963g (Period 0.56s)
BH-10	Amplification ratio	---------------------------	56.4 (f=1.9274Hz)
BH-1	Amplification ratio	---------------------------	34.9(f=1.9146Hz)
BH-10	Spectral Acceleration	---------------------------	26.1g (period 0.52s)
BH-1	Spectral Acceleration	---------------------------	8.22g (period 0.65s)

Table (7): Elastic half space Parameters

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

Discussion and Conclusion

This study tried to follow in conducting 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 site response study are summarized. A case study on ground response analysis of a site in Miyaneh city, Eastern Azarbayjan province of Iran, during the Ardabil earthquake (1997) 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 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 in the identification and to model the rupture mechanism at the source of an earthquake, evaluate the propagation of waves through the earth to the top of bed rock, determine 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 ground motions for earthquake resistant design of structures. Many earthquakes in the past have left many lessons to be learned 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. The results show a very detailed and stable V_s profile. The obtained V_s profile has a good comparative with other insitu 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 1.1g to 0.57g can use to prepare the PGA map of Miyaneh. They are not distributed uniformly due to variation in the soil profile at various locations. More than this PGA is comparable to the obtained peak horizontal acceleration values using SPT data .The shape of variation of peak acceleration with depth are similar to the SPT data. The calculated amplification factor ranged from 3.56 to 4.30 in elastic state and 34.9 to 56.4 in rigid condition can be used to prepare the amplification map of Miyaneh region.

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