Amplification based on shear wave velocity for seismic zonation: comparison of empirical relations and site response results for shallow engineering bedrock sites

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Abstract. Amplification based on empirical relations is widely used for seismic microzonation of urban centers. Amplifications are used to represent the site effects of a particular soil column. Many empirical correlations are available to estimate the amplification of seismic waves. These correlations are based on the ratio of shear wave velocity of foundation/rock to soil velocity or 30 m equivalent shear wave velocity (Vs^{30}) and are developed considering deep soil data. The aim of this work is to examine the applicability of available amplification relations in the literature for shallow engineering bedrock sites by carrying out site response studies. Shear wave velocity of thirteen sites having shallow engineering bedrock have been selected for the study. In these locations, the depth of engineering bedrock ($> 760 \pm 60$ m/s) is matched with the drilled bore hole. Shear wave velocity (SWV) has been measured using Multichannel Analysis of Surface Wave survey. These sites are classified according to the National Earthquake Hazards Reduction Program (NEHRP) classification system. Amplifications for an earthquake are arrived for these sites using empirical relations and measured SWV data. Site response analysis has been carried out in SHAKE using SWV and using synthetic and real earthquake data. Amplification from site response analysis and empirical relations are compared. Study shows that the amplification arrived using empirical relations does not match with the site response amplification. Site response amplification is much more than empirical values for same shear wave velocity.

Keywords: amplification; site response; Vs^{30}; earthquake; engineering bedrock.

1. Introduction

Soil condition modifies ground motion. In many cases, change in amplitude, frequency content and duration of ground motion are reported. Site-specific ground response analysis aims in determining the effect of local soil conditions, i.e. amplification of seismic waves. These are called as site effects, and are primarily based on geotechnical properties of the subsurface materials. Site effects are a combination of soil and topographical effects, which can modify (amplify and de-amplify) the characteristics (amplitude, frequency content and duration) of the incoming wave field. Induced effects may be liquefaction, land slide and/or Tsunami hazards. Amplification and liquefaction are the major effects of earthquakes that cause massive damage to the infrastructures

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and loss of lives. Recent study carried by United States Geological Survey (USGS) revealed that among deadly earthquakes reported for last 40 years, loss of lives and damages caused by ground shaking hazard was more than 80% of the total damages (Marano *et al*. 2010). Subsurface soil layers play a very important role in ground shaking damages. In many countries, seismic microzonation studies are carried out by considering local geotechnical properties up to 30 m depth by adopting seismic site characterization. Seismic site characterizations are carried out by considering geological data or soil conditions and strength of subsurface materials up to 30 m. TCEGE (1999) suggested empirical correlations for amplification estimation in Grade 2-Detailed Zonation. Relative amplification factors are correlated with the ratio of shear wave velocity of hard stratum/foundation to the average of soil shear wave velocity. Seismic site classification and amplification for seismic microzonation are obtained using average shear wave velocity in the top 30 m (Vs^{30}) irrespective of locations in Asia (Anbazhagan *et al*. 2010a). Anbazhagan *et al*. (2010a) showed that the 30 m based seismic site classification in the shallow engineering bedrock sites (engineering bedrock depth within 30 m) results in a stiffer site class and lesser spectral values. Authors also attempted site classification based on average SWV values up to the engineering bedrock. In this study seismic site classification and amplification factors are arrived using conventional approach and have been compared to site response results for sites having the engineering bedrock within 30 m. Shear wave velocities (SWV) measured in the shallow engineering bedrock region of Bangalore, India using Multichannel Analysis of Surface Wave (MASW) has been selected for the study. In these thirteen sites, rock depths obtained from SWV were matched with drilled borehole data. Engineering bedrock depth varies from 6 m to 27.6 m. These sites are classified based on conventional approach and amplification values are estimated considering empirical relations given in TCEGE (1999) and Kokusho and Sato (2008). Empirical correlations are available for relative amplification and average horizontal spectral amplification based on the shear wave velocity ratio or average 30 m SWV (Vs^{30}). Relative amplification is a ratio of peak velocity or peak acceleration of foundation to surface and related to ratio of foundation to soil velocity (TCEGE 1999 and Kokusho 2008). Average horizontal spectral amplification is defined as the ratio of average horizontal spectral values of the rock to surface in the period range of 0.4-2.0 seconds and related to 30 m average shear wave velocity (Borchert *et al*. 1991). Site response analysis has been carried out by giving input ground motion at engineering bedrock level and at 30 m depth. Earthquake data of synthetic ground motion for intra plate seismicity and recorded earthquake for inter plate seismicity is used for site response analysis. Synthetic ground motion for moment magnitude of 5.1 and peak ground acceleration (PGA) of 0.15 g generated for Bangalore, India has been used for intra plate seismicity. Chamoli real earthquake of moment magnitude 6.6 and PGA of about 0.2 g has been used for inter plate earthquake. Amplification estimated from empirical correlations are compared with the site response results and discussed in this paper.

2. Shear wave velocity using seismic survey

The geographic distribution of site classes based on Vs^{30} is useful for zonation studies because site amplification factor was defined as a function of Vs^{30} such that the effect of site conditions on ground shaking can be taken into account (Kockar *et al*. 2010). The site characterization for seismic microzonation is carried out by considering geological, geotechnical and geophysical data. The popular geotechnical tests are standard penetration test (SPT), dilatometer test (DMT), pressure meter test and seismic cone penetration test (SCPT). Among these, SPT is widely used in many countries because of abundant availability of existing data, technique and physical knowledge of soil and comparison with other tests. Many geophysical methods are attempted for seismic site characterization, but the methods widely used are Spectral Analysis of Surface Waves (SASW) and Multichannel Analysis of Surface Waves (MASW). MASW is a seismic surface wave method, widely used for sub-surface characterization and it is increasingly applied for seismic microzonation and site response studies (Anbazhagan and Sitharam 2008a and Anbazhagan *et al*. 2010b). In particular, MASW is used in geotechnical engineering to measure the shear wave velocity and dynamic properties of site (Anbazhagan and Sitharam 2008a, 2010). It is used to identify the sub-surface material boundaries, spatial and depth variations of weathered and engineering rocks (Anbazhagan and Sitharam 2009a). It is also used for the geotechnical characterization of near surface materials (Park *et al*. 1999, Xia *et al*. 1999, Miller *et al*. 1999, Kanli *et al*. 2006, Anbazhagan and Sitharam 2008b). The application of MASW is also extended to railway engineering to identify the degree and type of fouling (Anbazhagan *et al*. 2010c). First author carried out many MASW tests in India and Australia, out of which thirteen SWV profiles having engineering bedrock at shallow depth have been selected for this study. Locations of selected MASW data set are shown in Fig. 1(a) along with the locations of input earthquakes in India. MASW system consisting of 24 channels Geode seismograph with 24 vertical geophones of 4.5 Hz capacity are used to carry out the field experiments. A typical testing arrangement in the field is shown in Fig. 1(b). All tests have been carried out with geophone interval of 1 m. Source has been kept on both side of the spread

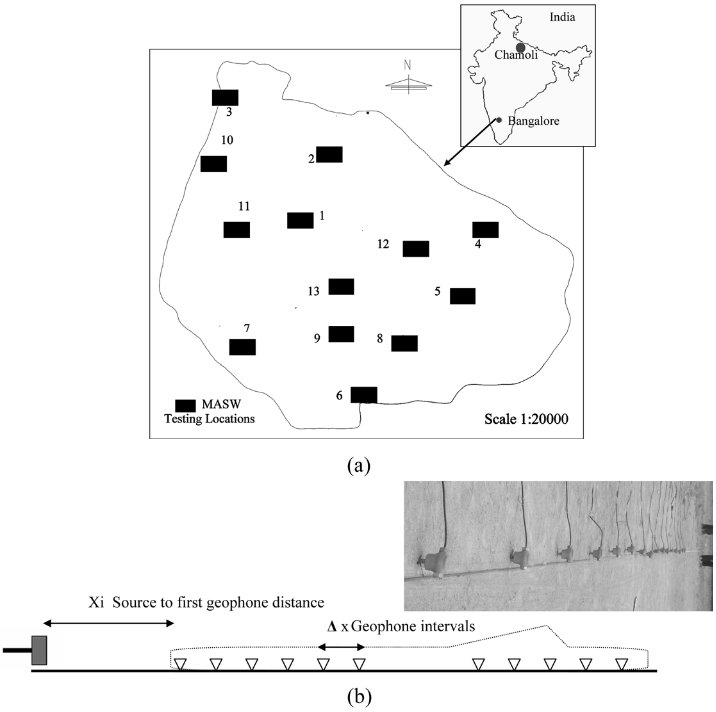


Fig. 1 (a) Locations of shear wave velocity profiles in Bangalore city with India map marked Chamoli and Bangalore and (b) typical linear array of geophones in the field for MASW survey and source to the first and last receiver were also varied from 5 m, 10 m and 15 m to avoid the effects of near-field and far-field. The seismic waves are created by impulsive source of 15 pound (sledge hammer) with 300 mm × 300 mm size hammer plate with ten shots. These waves are captured by 24 geophones/receivers of 4.5 Hz capacity. Fig. 2(a) shows typical surface wave record for location 6 (SWVP 6).

These captured data are further used to get dispersion curves, which are used to extract shear wave velocity at the midpoint of the testing locations. The shorter wavelengths are sensitive to the physical properties of surface layers (Xia *et al.* 1999). For this reason, a particular mode of surface wave will possess a unique phase velocity for each unique wavelength, leading to the dispersion of the seismic signal. For a multi-layered subsurface model, Rayleigh-wave dispersion curves can be calculated by Knopoff's method (Schwab and Knopoff 1972). Rayleigh-wave phase velocity, c_{Rj}, is determined by a characteristic Equation F in its nonlinear, implicit form

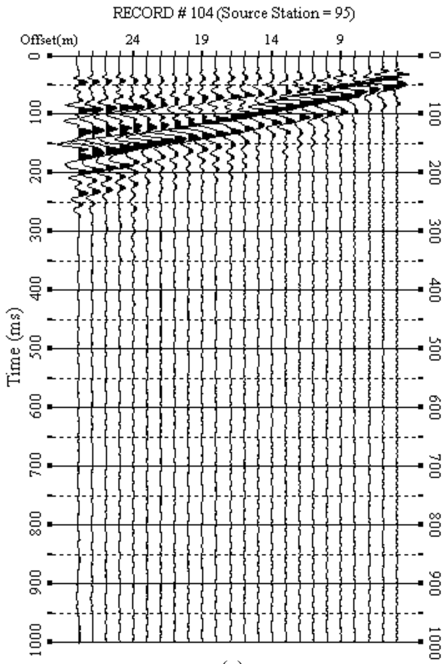
$$F(f_j, c_{Rj}, v_s, v_p, \rho, h) = 0 \qquad (j = 1, 2, \ldots, m) \tag{1}$$

where f_j is the frequency, in Hz; c_{Rj} is the Rayleigh-wave phase velocity at frequency f_j; $v_s = (v_{s1}, v_{s2}, \ldots, v_{sn})^T$ is the S-wave velocity vector, with v_{si} the shear-wave velocity of the i^{th} layer; n is the number of layers; $\mathbf{v}_p = (v_{p1}, v_{p2}, \ldots, v_{pn})^T$ is the compressive P-wave velocity vector, with v_{pi} the P-wave velocity of the i^{th} layer; $\rho = (\rho_1, \rho_2, \ldots, \rho_n)^T$ is the density vector, with ρ_i the density of the i^{th} layer; and $\mathbf{h} = (h_1, h_2, \ldots, h_{n1})^T$ is the thickness vector, with h_i the thickness of the i^{th} layer. Given a set of model parameters (v_s, v_p, ρ, and h) and a specific frequency (f_j), the roots of Eq. (1) are the phase velocities. If the dispersion curve consists of m data points, a set of m equations in the form of Eq. (1) can be used to find phase velocities at frequencies f_j ($j = 1, 2, \ldots, m$) using the bisection method (Press *et al.* 1992, Xia *et al.* 1999). In this study, only the fundamental mode is considered. The least analyzable frequency in this dispersion curve is around 10 Hz and highest frequency is 90 Hz. A dispersion curve of location 6 along with signal amplitude and signal to noise ratio is shown in Fig. 2(b).

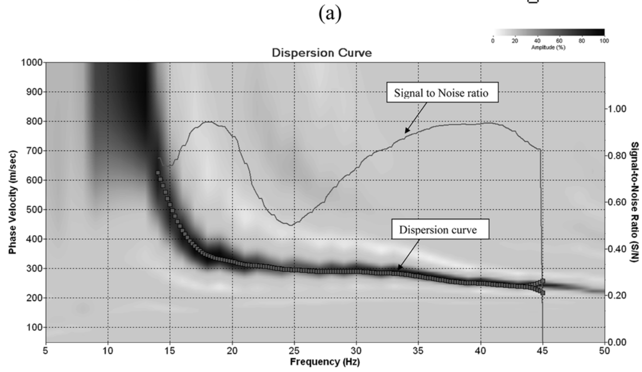
Shear wave velocity can be derived from inverting the dispersive phase velocity of the surface (Rayleigh and/or Love) wave (Dorman and Ewing 1962, Aki and Richards 1980, Mari 1984, Xia *et al.* 1999). Shear wave velocity profile was calculated using an iterative inversion process that requires the dispersion curve developed earlier as input. A least-square approach allows automation of the process (Xia *et al.* 1999). S-wave velocities of each layer can be represented as the elements of a vector $\mathbf{x}$ of length n, or $\mathbf{x} = [v_{s1}, v_{s2}, v_{s3}, \ldots, v_{sn}]^T$. Similarly, the measurements (data) of Rayleigh-wave phase velocities at m different frequencies can be represented as the elements of a vector $\mathbf{b}$ of length m, or $\mathbf{b} = [b_1, b_2, b_3, \ldots, b_m]^T$. Since the model $\mathbf{c}_R$ (Eq. (1)) is a nonlinear function, Eq. (1) must be linearized by Taylor-series expansion to employ the matrix theory

$$J\Delta X = \Delta b \tag{2}$$

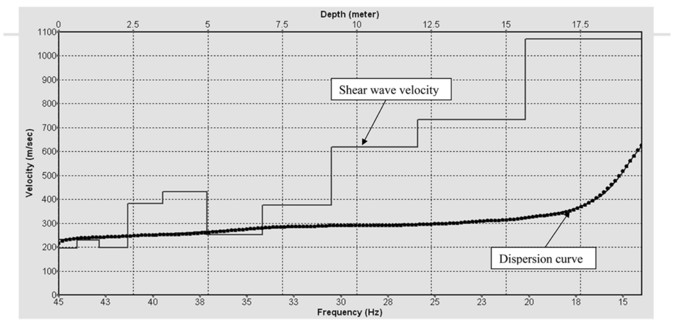
where $\Delta\mathbf{b} = \mathbf{b} - \mathbf{c}_R(\mathbf{x}_0)$ and is the difference between measured data and model response to the initial estimation, in which $\mathbf{c}_R(\mathbf{x}_0)$ is the model response to the initial S-wave velocity estimates, $\mathbf{X}_0$; $\Delta\mathbf{X}$ is a modification of the initial estimation; and $\mathbf{J}$ is the Jacobian matrix with m rows and n columns ($m > n$). The elements of the Jacobian matrix are the first-order partial derivatives of $\mathbf{c}_R$ with respect to S-wave velocities. Since the number of data points contained in the dispersion curve is generally much larger than the number of layers used to define the subsurface ($m > n$), Eq. (1) is



(a)



(b)



(c)

Fig. 2 (a) Seismic record of signals in MASW system for location 6, (b) typical dispersion curve obtained from MASW survey for location 6 and (c) typical shear wave velocity profile obtained for location 6 usually solved by optimization techniques. The objective function can be defined as

$$\Phi = \|J\Delta X - \Delta b\|_2 W \|J\Delta X - \Delta b\|_2 + \alpha \|\Delta X\|_2^2 \tag{3}$$

where $\| \ \|_2$ is the l_2-norm length of a vector, α is the damping factor, and **W** is a weighting matrix. This is a constrained (weighted) least-squares problem. More details about the sensitivity of each parameter and calculation with respective examples are detailed in Xia *et al*. (1999). Shear wave velocities of each location were inverted from respective dispersion curves. Shear wave velocity profile obtained for location 6 is shown in Fig. 2(c). Shear wave velocity obtained from this technique is comparable with cross hole, up and down hole seismic method with error of 8 to 15% (Park *et al*. 1999). Thirteen selected shear wave velocity profiles for the present study are shown in Figs. 3(a) and 3(b). It can be observed that layer having SWV of 400 m/s to 700 m/s is within 30 m in all the profiles. This layer is considered as a base layer for engineering design or to give design input motions. MASW tests considered in this study are close to SPT boreholes, where soil layers and rock depth are comparable (Anbazhagan and Sitharam 2009a).

3. Rock depth and average shear wave velocity

Geotechnical characteristics of soil deposits play an important role in the level of ground shaking or local site effects. Ground classification of individual sites based on soil boring data or SWV is a more direct indicator of local site effects. Site effects in terms of amplification at soil sites require knowledge of shear stiffness of the soil column, expressed in terms of SWV (Borcherdt 1994). The site classes are defined in terms of SWV up to a depth of 30 m, denoted by Vs^{30}. If no measure-

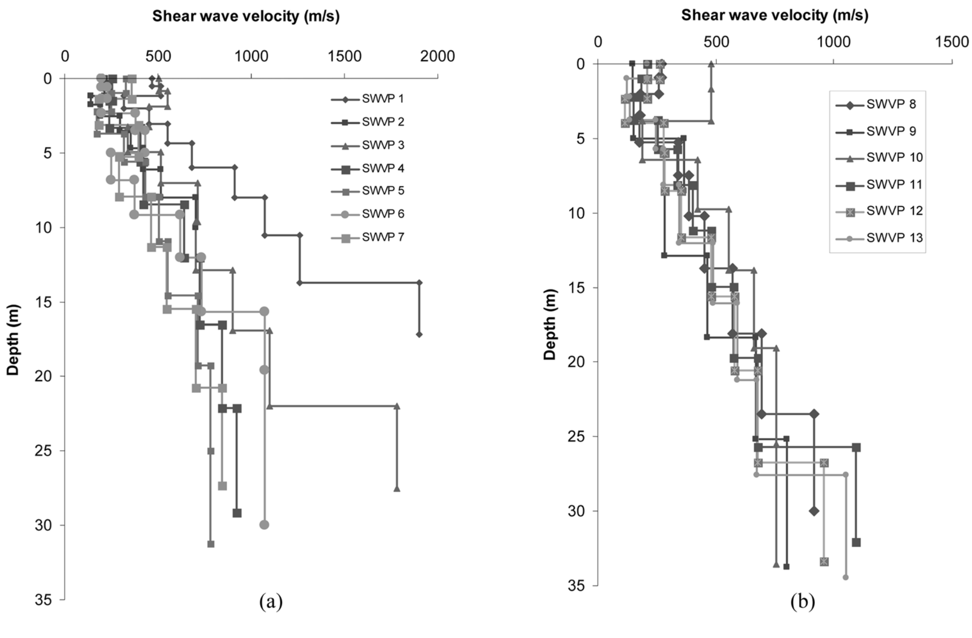


Fig. 3 Selected shear wave velocity profile with shallow engineering bedrock

Table 1 Average shear wave velocity, rock depth and site classification of selected sites

Site	Depth weathered rock	Soil average SWV [Vs_{330}] (< 330 ± 30 m/s)	Vs_b/Vs_{330}	Depth engineering rock	Soil average SWV [Vs_{760}] (< 760 ± 60 m/s)	Vs_b/Vs_{760}	Vs^{30}	Site class as per NEHRP and IBC 2009
SWVP 1	2.7	434.5	1.04	6.0	498.2	1.83	1081.6	Site class B
SWVP 2	4.7	216.6	1.63	10.0	287.6	2.44	512.8	Site class C
SWVP 3	1.0	462.2	1.12	12.9	473.4	1.51	802.0	Site class B
SWVP 4	5.5	247.8	1.65	16.5	392.9	1.84	569.1	Site class C
SWVP 5	7.0	250.6	1.27	19.2	365.6	1.95	494.2	Site class C
SWVP 6	3.5	208.4	1.83	15.7	335.5	2.19	553.4	Site class C
SWVP 7	10.0	306.5	1.51	20.8	372.9	1.89	492.5	Site class C
SWVP 8	10.2	221.9	1.54	23.5	370.5	2.47	431.3	Site class C
SWVP 9	16.5	237.2	1.96	25.2	347.6	2.31	346.6	Site class C
SWVP 10	9.3	384.2	1.10	25.5	465.8	1.62	505.3	Site class C
SWVP 11	8.2	199.6	1.69	25.7	363.6	3.00	410.5	Site class C
SWVP 12	11.7	230.5	1.54	26.7	360.3	2.65	382.2	Site class C
SWVP 13	12.0	199.2	1.73	27.6	302.4	3.48	362.2	Site class C

ments of SWV to 30 m are feasible, standard penetration resistance (N_{30}) and undrained shear strength (S_u^{30}) can be used (Borcherdt 1994). Equivalent SWV values for 30 m depths (Vs^{30}) are followed for site classification in the National Earthquake Hazards Reduction Program (NEHRP) recommendation (The Building Seismic Safety Council BSSC 2001) and also International Building Code (IBC) classification (IBC 2009) (Dobry *et al.* 2000, Kanli *et al.* 2006). To classify the study area, the shear wave velocity ranges suggested by NEHRP (BSSC 2001) are, site class A ($Vs^{30} >$ 1.5 km/s), site class B (0.76 km/s < Vs^{30} = 1.5 km/s), site class C (0.36 km/s < Vs^{30} = 0.76 km/s), site class D (0.18 km/s < Vs^{30} = 0.36 km/s) and site class E (Vs^{30} < 0.18 km/s) have been considered. Vs^{30} has been calculated for selected sites and classified according to NEHRP and IBC. Vs^{30} and respective site class for selected sites are given in Table 1. It is observed that site 1 and 3 comes under site class B with no amplification and remaining sites comes under site class C where amplification is minor. Hard stratum or rock depth and shear wave velocities are important parameter to estimate amplification. Two types of rocks can be identified at any location for engineering applications; one is weathered rock having shear wave velocity of about 330 ± 30 m/s. Another one is engineering bedrock having SWV of about 760 ± 60 m/s (Anbazhagan and Sitharam 2009a). In contrast, upper crust SWV of 3000 to 3600 m/s is used for seismological research in geophysics. Kokusho (2008) and Kokusho and Sato (2008) have calculated the equivalent surface layer velocity by considering thickness and the first and higher order peaks from recorded earthquake data main and aftershocks. They have also suggested an equivalent surface layer velocity, which may be calculated using dominant frequency from micro-tremor and borehole data. But in this study soil equivalent velocity is estimated by considering measured SWV to compare the amplification, because of limited recorded ground motion and dominant frequency for selected sites. Average soil velocities are calculated by averaging shear wave velocity of layers upto layer with SWV below 330 ± 30 m/s and 760 ± 60 m/s. Average shear wave velocity upto weathered and engineering rock are denoted as Vs_{330} and Vs_{760}. Depth of weathered and engineering rock along with average shear wave velocity up to these rock layers are calculated and presented in Table 1. In

Table 1, it can be noticed that few profiles Vs_{330} values are more than 330 m/s, which is attributed due to the presence of high velocity layers (more than 360 m/s) above the weathered rock layers. Borehole study carried out by Anbazhagan and Sitharam (2009a) shows that top layer (high velocity) is not a weathered rock layer, but layer with SWV of 330 ± 30 m/s is noticed below higher velocity layers which matches well with the borehole weathered rock information. Hence average SWV calculated up to the layer having SWV of 330 ± 30 m/s, because of this Vs_{330} is more than 330 m/s. Average velocity upto hard stratum/ weathered rock and engineering rock is much less than Vs^{30}. This difference is more if engineering bedrock depth is less than 20 m and less for above 20 m. It reveals that Vs^{30} in shallow bedrock sites included rock velocity and not representing soil average SWV. The ratio of shear wave velocity for foundation layers (weathered rock/engineering rock) to soil shear wave velocity has been calculated which is also given in Table 1. These values are useful for estimation of amplification at later stages.

4. Amplification correlations

Soil stiffness in the form of shear wave velocity is a useful parameter and widely used to indicate/ estimate the amplification of seismic waves. Shima (1978) developed a linear relation between amplification and ratio of shear wave velocity of bedrock to soil (V_{SR}) using multiple reflection theory. Author used 816 points to relate mean amplification magnitudes and velocity ratio. Author used SWV profiles having average shear wave velocity of less than 250 m/s and maximum layer velocity of 600 m/s. He also highlighted that the derived relation does not account for thickness and number of layers. It has been noticed that the soil thickness was about 50 m. Fig. 4 shows the relation generated by Shima (1978) and respective correlation obtained for amplification magnitude (AM) based on Shima (1978) study is given below

$$AM_L = 1.87 + 1.5V_{SR} \text{ for } largest \text{ max} \tag{4}$$

$$AM_{FS} = 1.95 + 1.33V_{SR} \text{ for } first \text{ max} \tag{5}$$

Where, AM_L and AM_{FS} are amplification magnitudes corresponding to the largest and first maximum of predominant responses. V_{SR} is the shear wave velocity ratio of basement/foundation and surface layers. Joyner and Fumal (1984) related the peak velocity amplification (PVA) to average shear wave velocity over a depth of a one-quarter wavelength for a one second period wave (in m/s). Relative amplification factor (A) for peak ground velocity by Joyner and Fumal (1984) is given below

$$A = 23(V_{OQW})^{-0.45} \tag{6}$$

Where V_{OQW} is the average shear wave velocity over a depth of one-quarter wavelength for one-second wave (in m/s). This relation is not widely used because of limited source of information and difficulty to estimate the average shear wave velocity for one quarter wave. Midorikawa (1987) presented a relation between peak velocity amplification (PVA) factors to 30 m average shear wave velocity. Midorikawa (1987) is published in Japanese, so it is difficult to access the technical content. However these relations are tabulated in manual for zonation on seismic geotechnical hazards published by Technical Committee for Earthquake Geotechnical Engineering (TCEGE

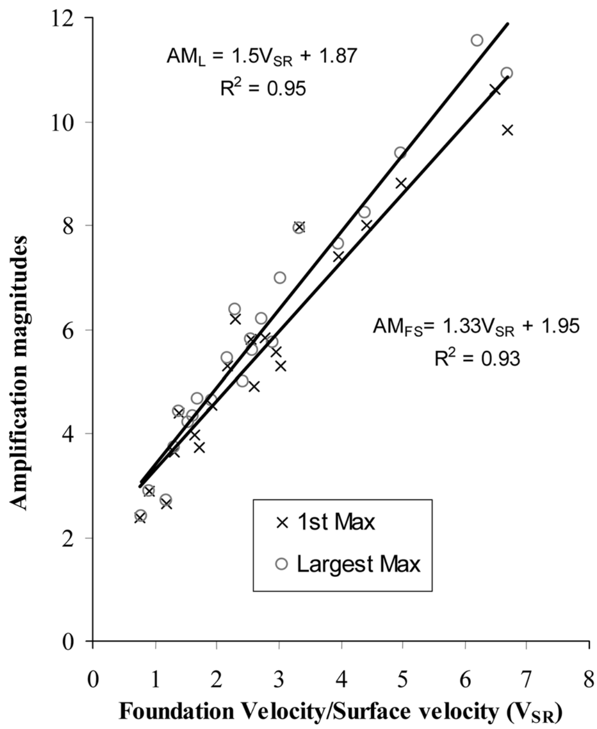


Fig. 4 Relation between the shear wave velocity ratio of surface and the basement formations and the amplifications magnitudes (modified after Shima 1978). 1st max and largest max mean a first and largest maximum amplification of response

1999), which are given below

$$A = 68(Vs^{30})^{-0.6} \ldots (Vs^{30} < 1100\ m/s) \tag{7}$$

$$A = 1 \ \ldots (Vs^{30} > 1100\ m/s) \tag{8}$$

Relative amplification factor developed by Midorikawa (1987) and Joyner and Fumal (1984) are compared in TCEGE (1999), which shows that both are close after the average shear wave velocity of above 700 m/s i.e. rock. Borchert *et al*. (1991) presented a relation between average horizontal spectral amplification in the period range of 0.4-2.0 sec to 30 m mean shear wave velocity. Average horizontal spectral amplification (AHSA) is in the period range of 0.4-2.0 sec for weak and strong motions (Borchert *et al*. 1991 and TCEGE 1999)

$$AHSA = 700/Vs^{30}(\text{for weak motion}) \tag{9}$$

$$AHSA = 600/Vs^{30}(\text{for strong motion}) \tag{10}$$

These methods were also presented in TCEGE (1999) for Grade-2 seismic microzonation to estimate the amplification factor. The relative amplification and average horizontal spectral amplification factors are related to the 30 m average values and are widely used for microzonation studies. Recently Kokusho and Sato (2008) and Castellaro *et al*. (2008) highlighted that Vs^{30} is a weak proxy to seismic amplification. Kokusho (2008) and Kokusho and Sato (2008) have estimated amplification using strong motion records at surface and base layers in the KiK-net arrays of 3 strong earthquakes in Japan. They have calculated the average S-wave velocity ($\overline{Vs}$) for equivalent surface layers corresponding to each peak of Fourier spectrum using S-wave logging. Authors obtained the ratio of base layer SWV (Vs_b) to equivalent surface layer velocity ($\overline{Vs}$) ($Vs_b/\overline{Vs}$) and related to amplification. Authors have used downhole seismometer measured data for a depth of 100 m to 330 m and considered base layer velocity (Vs_b) of 400 m/s to 3000 m/s. Amplification ($2A_s/2A_b$) can be calculated using the following equation

$$2A_s/2A_b = 0.175 + 0.685(Vs_b/\overline{Vs}) \tag{11}$$

Where, A_s is the Fourier spectrum for incident wave at the ground surface and A_b is the Fourier spectrum for the base layer. This correlation is only applicable under the condition of $Vs_b/\overline{Vs} \leq 10$. Kokusho and Sato (2008) highlighted that conventionally followed Vs^{30} and Vs_b/Vs^{30} for seismic microzonation does not fit well and a parameter more site dependent is preferable. Fig. 5 shows the comparison of amplification based on the ratio of base layer velocity to soil layer velocity presented by Shima (1978) and Kokusho (2008). Both the studies were carried out in Japan but amplification predicted by Shima (1978) is 3.5 times more than that of Kokusho (2008) for lower velocity ratio i.e. harder materials, and 2.25 times more for higher velocity ratio i.e. soft materials above base layers. Similarly, Fig. 6 shows the comparison of relative amplification and AHSA based on Vs^{30} based on Midorikawa (1987) and Borchert *et al.* (1991). Relative amplification presented by Midorikawa (1987) is about 50% less than AHSA presented by Borchert *et al.* (1991) for same Vs^{30}.

5. Site response analysis of shallow engineering bedrock sites

Site effect studies and amplification correlations with shear wave velocity are available for deep soil sites where the engineering bedrock is not noticed or more than 100 m. In contrast, limited

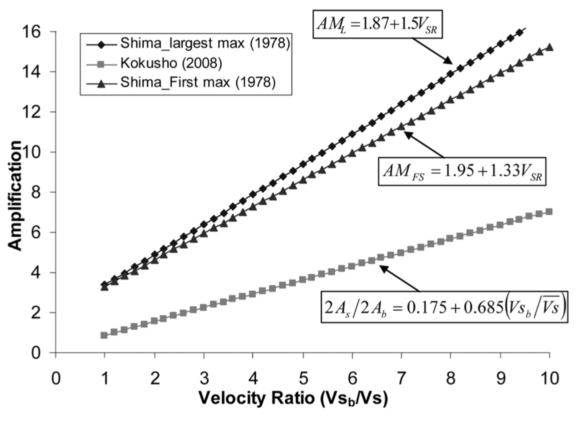


Fig. 5 Amplification relations developed by Shima (1978) and Kokusho (2008) considering shear wave velocity ratio of basement to soil

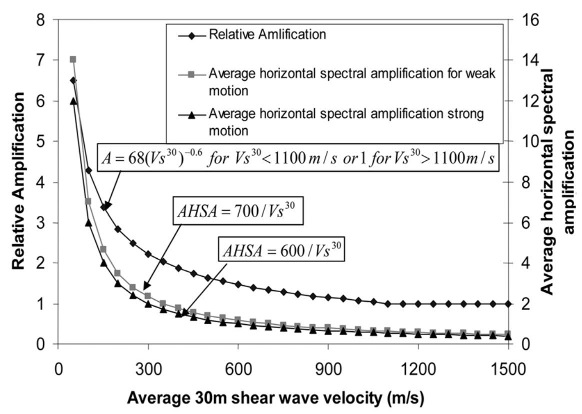


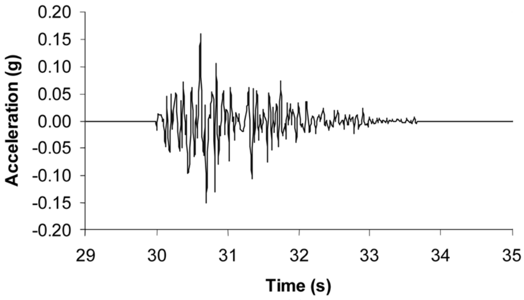
Fig. 6 Amplification relations developed by Midorikawa (1987) and Borchert *et al.* (1991) considering average 30 m shear wave velocity

recorded ground motions at rock and surface with shear wave velocity profiles are available for shallow rock sites. The correlations developed for deep soil sites are used directly to represent the site effects in the seismic microzonation irrespective of engineering bedrock depth in the region. These practices are widely followed by many researchers because it is given in the manual for zonation on seismic geotechnical hazards published by Technical Committee for Earthquake Geotechnical Engineering (TCEGE 1999). It is obvious to verify these correlations suitability by measured amplification data from shallow engineering bedrock regions. Due to lack of recorded site response parameters from the shallow engineering bedrock sites to authors, in this study site response analysis carried out using site response program of SHAKE. The computer program SHAKE was written in 1970-71 by Dr. Schnabel and Prof. John Lysmer (Schnabel *et al*. 1972). This program is used for computing seismic response of horizontally layered soil deposits. One-dimensional ground response analysis using equivalent linear model has been carried out using SHAKE 2000 software in which ground motion of the object can be given in any one layer in the system and motions can be computed in any other layer. In equivalent linear approach, the non-linearity of the shear modulus and damping is accounted for the use of equivalent linear soil properties using an iterative procedure to obtain values for modulus and damping compatible with the effective strains in each layer. In this study, the degradation curves given by Seed and Idriss (1970) and Schnabel (1973) for sand average and rock has been used for soil and rock layers respectively.

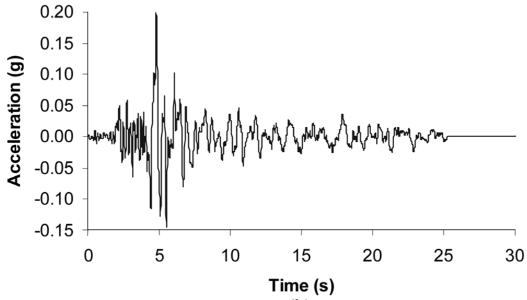
6. Input ground motion

Many large deadly earthquakes with different magnitudes have occurred in Asia particularly in India and China. But limited acceleration time histories are available for researchers to understand the behavior of site effects. For the area having poor seismic record, synthetic ground motion model is an alternative (Sitharam and Anbazhagan 2007). Iyengar and RaghuKanth (2004) and RaghuKanth and Iyengar (2007) have developed ground motion attenuation relation based on the statistically simulated seismological model. Modeling of strong motion helps to estimate the future hazard of the region and study the local effects in local scale. Seismological model by Boore (1983 2003) is used for generation of synthetic acceleration-time response (Atkinson and Boore 1995, Hwang and Huo 1997). In order to understand the site effects due to moderate earthquake, synthetic ground motion generated by Sitharam and Anbazhagan (2007) and Anbazhagan and Sitharam (2009b) has been used in this study. This synthetic ground motion was developed by considering the active lineament source of Mandya-Channapatna-Bangalore lineament and reported the past earthquake of 5.1 in moment magnitude close to this source. Epicentral distance of 5.2 km and focal depth of 10 km in the seismic source location of crustal rock with SWV of 3.65 km/s and density of 2.75 g/cc was considered for synthetic ground motion model (Sitharam and Anbazhagan 2007).

Fig. 7(a) shows the synthetic ground motion used as input. This synthetic ground motion having peak acceleration of 0.155 g for moment magnitude of 5.1 and represents intra plate seismic characteristic. Typical inter plate earthquake reported in Chamoli have been taken from Atlas of Indian Strong Motion Records (Shrikhande 2001). The Chamoli earthquake occurred on 29 March 1999 at north of Chamoli in the Lesser Himalayas. This event had a moment magnitude of 6.6 and peak acceleration of 0.199 g recorded at rock level. Focal depth of the event is 15 km. This event is recorded by Department of Earthquake Engineering, Indian Institute of Technology, Roorkee at



(a)



(b)

Fig. 7 Input ground motion used for study: (a) synthetic ground motion for *Mw* of 5.1-Intra plate earthquake and (b) recorded rock motion at Chamoli, *Mw* of 6.6-Inter plate earthquake

GOPESHWAR station. The station is located at hypocentral distance of 17.3 km. Fig. 7(b) shows the acceleration time history of Chamoli earthquake used in this paper.

7. Results and discussion

Site response analysis has been carried out by giving input ground motions at engineering bedrock level where SWV is more than 760 ± 60 m/s and at 30 m depth. For 30 m site response analysis SWV and density of last layer extends up to 30 m according to Boore (2004) for Vs^{30} calculation and site response analysis. Peak ground acceleration (PGA) with depth, response spectrum for rock and surface layers, Fourier spectrum and amplification ratio are arrived. To focus on amplification calculation, only PGA and response spectrum has been considered here. Fig. 8 shows the spectrum at rock for input motion of Bangalore earthquake along with the surface spectra obtained by giving input at engineering bedrock (ER) and 30 m depth. Zero spectral acceleration (PGA at zero periods) value of 0.70 g is obtained for input at ER and 0.88 g is obtained for input at 30 m. Peak spectral values are almost comparable for both the cases, but period corresponding to peak spectral value for input at 30 m is less than the input at ER. Similarly Fig. 9 shows response spectrum of input motion of Chamoli earthquake along with surface spectra obtained by giving input at engineering

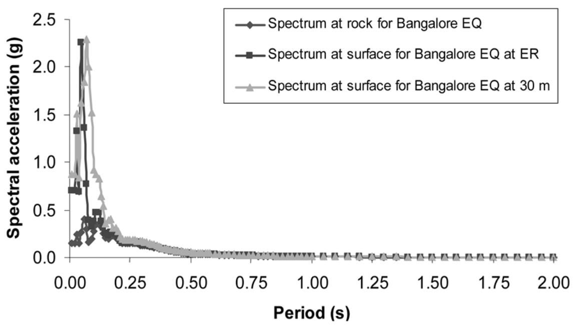


Fig. 8 Typical response spectrum at rock and surface for Bangalore earthquake at engineering bedrock and 30 m

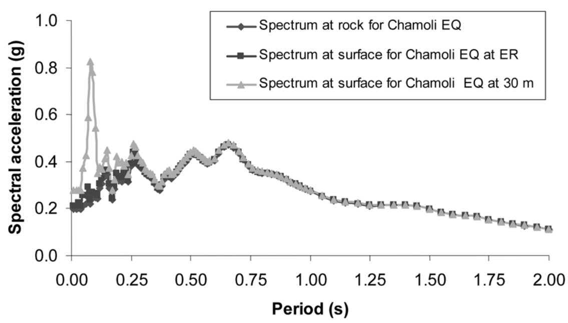


Fig. 9 Typical response spectrum at rock and surface for Chamoli earthquake at engineering bedrock and 30 m

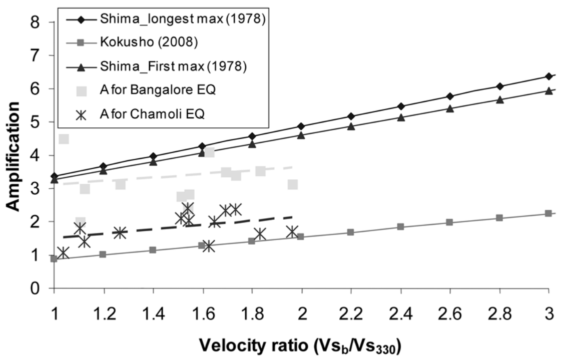


Fig. 10 Relative amplification (peak surface by peak rock acceleration) versus SWV ratio of basement layer SWV > 330 ± 30 m/s to average shear wave velocity up to layer of SWV > 330 ± 30(EQ-earthquake)

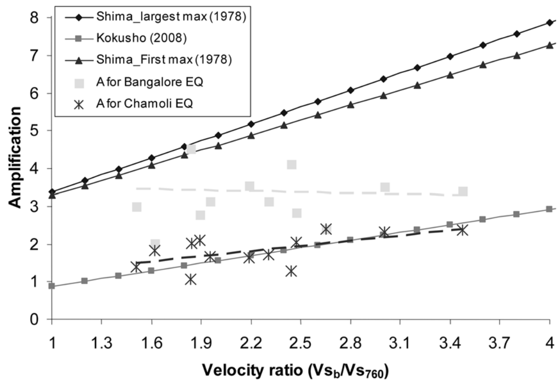


Fig. 11 Relative amplification (peak surface by peak rock acceleration) versus SWV ratio of basement layer SWV > 760 ± 60 m/s to average shear wave velocity up to layer of SWV > 760 ± 60 (EQ-earthquake)

bed rock and 30 m depth. Zero spectral acceleration value of 0.21 g is obtained for input at ER and 0.28 g is obtained for input at 30 m. Peak spectral value and period corresponding to peak spectral value for the input at 30 m is more than the input at ER. Further relative amplification ratios (PGA surface by PGA rock) have been calculated for intra plate (Bangalore) and inter plate (Chamoli) earthquakes separately. These relative amplifications are compared with empirical relations discussed above. Fig. 10 shows reproduced Fig. 5 with relative amplification from site response versus ratio of foundation SWV ($Vs_b > 330 \pm 30$ m/s) to average soil SWV (Vs_{330}). Amplification arrived using empirical correlations does not match with the amplification from response analysis. Amplification due to Chamoli earthquake is relatively close to Kokusho (2008) relation but larger than proposed values. Fig. 11 shows reproduced Fig. 5 with relative amplification from site response versus ratio of foundation SWV ($Vs_b > 760 \pm 60$ m/s) to average soil SWV (Vs_{760}). Amplification arrived using empirical correlations does not match with amplification from the response analysis for Bangalore earthquake. Amplification due to Chamoli earthquake is relatively comparable with amplification from Kokusho (2008) relation. It can be noted here that Kokusho (2008) and Kokusho and Sato (2008) have considered basement velocity of 400 m/s to 3000 m/s for sites having depth more than 100 m soil. Mismatching may be attributed by empirical correlations, which are developed considering deep soil profiles and larger earthquake magnitudes. Fig. 12 shows the amplification from empirical relation based on Vs^{30} with amplification from site response analysis by giving input at 30 m. Practically in many cases, earthquake input will be given at rock where Vs is more than 700 m/s, but in this study to check the amplification, the input is given at 30 m. The amplification obtained from site response analysis does not match with the empirical amplification given by Midorikawa (1987) for Bangalore and Chamoli earthquake, except for two values from Chamoli earthquake. Empirical amplification recommended in zonation on seismic geotechnical hazards published by Technical Committee for Earthquake Geotechnical Engineering (TCEGE 1999) may not be directly applicable for regions with shallow engineering bedrock. In these regions, the amplification should not be estimated using TCEGE (1999) recommendations.

8. Average horizontal spectral amplification

Response spectrum at top layer of each location has been arrived from site response analysis. The average horizontal spectral values of rock and top layers have been estimated for the period range of 0.4-2.0 s. Average horizontal spectral amplification (AHSA) has been estimated for each location for Bangalore and Chamoli earthquakes separately by giving the input at engineering rock level and 30 m. Fig. 13 shows AHSA from site response analysis by giving input at engineering bedrock level along with the empirical relation given by Borchert *et al*. (1991). AHSA arrived from site response study does not match with Borchert *et al*. (1991) relation. AHSA due to Bangalore earthquake follow a trend similar to Borchert *et al*. (1991) but values are much higher. Six AHSA values obtained from input of Chamoli earthquake are closer to Borchert *et al*. (1991) strong motion correlation (Eq. (5)), but overall amplification trend is different from Borchert *et al*. (1991). Fig. 14 shows AHSA from site response analysis arrived at by giving input at 30 m along with the empirical relation given by Borchert *et al*. (1991). AHSA due to Bangalore earthquake follow a trend similar to Borchert *et al*. (1991), but values are much higher, which are similar to the input at engineering rock level results. AHSA values obtained from input of Chamoli earthquake is close to Borchert *et al*. (1991) strong motion correlation (Eq. (5)) for Vs^{30} of 400 to 550 m/s. AHSA values obtained by giving input at engineering bedrock level is more than the input at 30 m, but differences are small. Fig. 15 shows AHSA from Bangalore and Chamoli earthquake by giving input at engineering bedrock level and 30 m. AHSA obtained from Chamoli earthquake is much lower than that of Bangalore earthquake. This may be due to synthetic and recorded earthquake inputs, but it is difficult to comment based on these two input earthquake analysis. AHSA obtained by giving input at 30 m is much higher than input at engineering bedrock level in both events. Input at engineering rock depth and 30 m shows closer amplification values when average 30 m shear wave velocity about 400 m/s (See Fig. 15) and other Vs^{30} amplification from input at engineering bed rock is less than the amplification from input at 30 m.

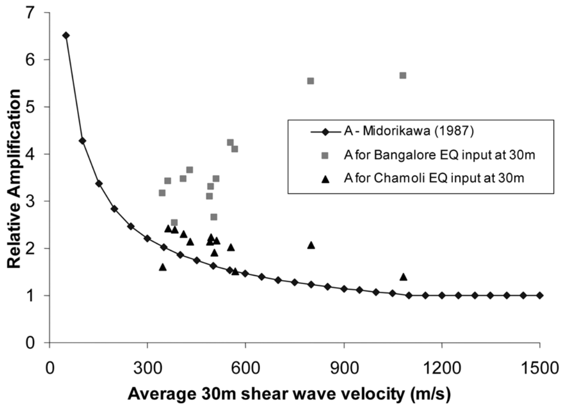


Fig. 12 Amplification obtained from Midorikawa (1987) compared with amplification from site response analysis for Bangalore and Chamoli earthquakes (EQ-earthquake)

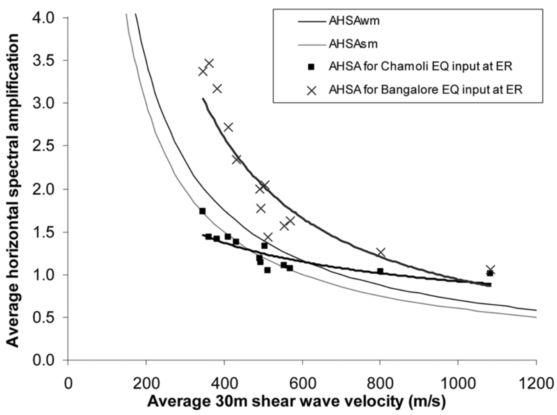


Fig. 13 Average horizontal spectral amplification from site response analysis by giving input at engineering bedrock level compared with Borchert *et al*. (1991) relations (EQ-earthquake, AHSAwm-weak motions and AHSAsm-strong motion)

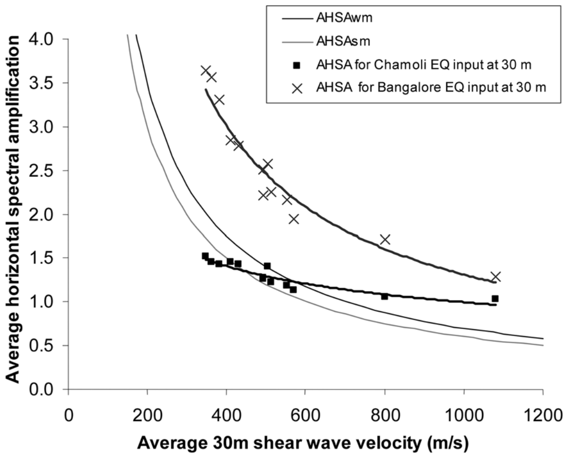


Fig. 14 Average horizontal spectral amplification from site response analysis by giving input at 30 m compared with Borchert *et al*. (1991) relations (EQ-earthquake, AHSAwm-weak motions and AHSAsm-strong motion)

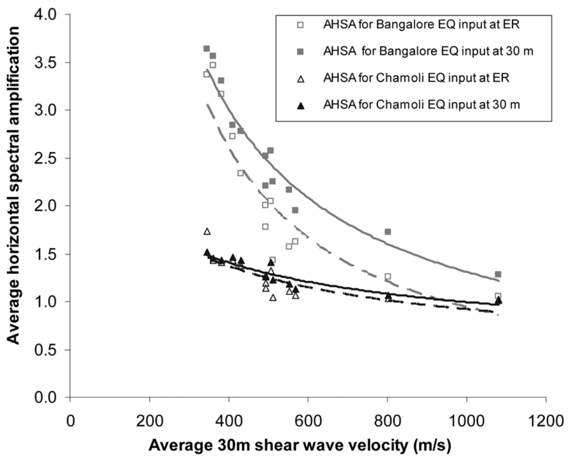


Fig. 15 Average horizontal spectral amplification from site response analysis for Bangalore and Chamoli earthquakes by giving input at engineering bedrock and 30 m level (EQ-earthquake)

This study shows that direct application of empirical relation listed in literature for microzonation to estimate amplification in shallow engineering bedrock sites is limited. Amplification is site specific with respect to depth of engineering bedrock and SWV of soil and region specific with respect to earthquake source mechanism. So empirical relations developed by considering deep soil sites in other regions may not be directly used in the region with shallow engineering bedrock. These also concur with finding by Anbazhagan *et al*. (2010a), i.e. following 30 m approach to classify the site in shallow engineering bedrock region results in stiffer site class and lower spectral accelerations. These sites are classified as site class B and C as per NEHRP and IBC, but study shows the significant amplification due to impedance contrast between shallow weak soils followed by hard rock. Many researchers have raised doubts about validity of Vs^{30} to soil amplifications in tectonically active regions (Wald and Mori 2000, Mucciarelli and Galipoli 2006, Castellaro *et al*. 2008, Kokusho 2008). A new site classification system may be warranted for shallow engineering bedrock sites with amplification and spectral ratio correlations.

9. Conclusions

This paper presents the summary of empirical relations used for amplification estimation in microzonation studies. The empirical relations are based on the ratio of foundation/hard layer velocity to soil velocity and average 30 m velocity using data from deep soil profiles. Site classification and amplification are conventionally arrived based on these approaches. But 30 m average SWV is much more than soil average SWV when engineering bedrock depth is less than 25 m. This 30 m approach also results in stiffer site class and less spectral values. Site response analysis has been carried out using measured shear wave velocity from shallow engineering bedrock region (rock depth within 30 m) and intra plate earthquake suitable to Bangalore and inter plate earthquake from Chamoli. Earthquakes are assigned at engineering bedrock and 30 m level. This study shows that amplification from empirical relations does not match with the amplification from the site response results. Amplification empirical relations developed by Shima (1978), Joyner and Fumal (1984), Midorikawa (1987) and Borchert *et al*. (1991) are given in the zonation on seismic geotechnical hazards published by Technical Committee for Earthquake Geotechnical Engineering are over predict if a velocity ratio is considered and under predict if Vs^{30} is considered for same sites. It is also observed that average horizontal spectral amplification based on Vs^{30} does not match with site response results by giving input at engineering bedrock and 30 m. Application of empirical correlations to estimate the amplification for zonation studies may not directly represent the site effects of sites having engineering bedrock within 25 m. Amplification obtained by giving input at 30 m is higher than by giving input at engineering rock level. Recorded inter plate earthquake of Chamoli shows less amplification when compared to synthetic intra plate earthquake of Bangalore. Amplification obtained from site response study does not follow any trend with the velocity ratio and Vs^{30} as shown in literature, which may be attributed by depth of soil column. Empirical correlations developed by Shima (1978), Midorikawa (1987), Borchert *et al*. (1991) and Kokusho and Sato (2008) are considered to be deep soil sites data, but in this study soil thickness are within 30 m. AHSA from Bangalore earthquake follows a trend, but AHSA values are much higher than those from empirical correlations. AHSA due to Chamoli earthquake does not follow any trend but few AHSA values matches with old AHSA relation for Vs^{30} of about 400. Amplification relations available in the literature may not be directly applicable to the shallow engineering bedrock sites.

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