



Site Response Study and Amplification Factor for Shallow Bedrock Sites

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Abstract Amplification is a key parameter considered to account modification of seismic wave in the soil for earthquake resistance design of structure placed on soil. Initially, earthquake wave amplifications are related with shear wave velocity (V_s) ratio of soil and foundation layer, and then it was related to average value of V_s up to 30 m (V_s^{30}). Application of V_s^{30} concept to represent amplification in shallow bedrock sites is questionable and has rock velocity added to soil velocity. In this study, shallow bedrock sites in Bengaluru, Chennai, Coimbatore and Vishakhapatnam has been analyzed to understand amplifications and its proximity to V_s^{30} . The site response calculations are done using one-dimensional non-linear approach. Intraplate recordings from around the world suitable for the study area are selected. For comparison with known data, the amplification factors are evaluated considering the period ranges similar to international codes. Then, considering spectral signatures from response spectra for all site rock and surface motions, the factors are calculated for period range 0.01–0.06 s and 0.05–1.0 s. Based on the study, only one mid-period amplification factor has been proposed. This amplification factor represents the significant amplification of the site. Acceleration spectra show similar trends for different site classes irrespective of the fact that profiles are selected based on V_s values or SPT-N values. This is observed over a wide range of results and indicates shear modulus as an

influencing parameter. Comparison of spectral signatures for different site classes suggests that amplification reduces as the modulus of the soil column increases. Thus, it may be appropriate to classify sites based on shear modulus of soil column.

Keywords Site response · Amplification factors · Shallow bedrock · Seismic site classification · Shear modulus · Peninsular India

Introduction

Site response studies are carried out to estimate the modification in ground motion parameters due to local soil conditions when subjected to transient conditions. In general, site period and stiffness of soil column are the most important factors influencing the site response analysis. Site response analysis is performed using frequency-domain equivalent linear analysis or time-domain nonlinear analysis employing nonlinear hysteretic soil models.

For classifying a site, the defining parameters are those that affect site response most and are conveniently measurable. As shear stiffness and the time period have most effect on soil response, different classification systems followed worldwide [1–5] are based on parameters as shear velocity V_s and SPT-N. [1–4] use V_s^{30} and N^{30} to classify, while [5] employs V_s^{20} . Indian Standard IS 1893 uses SPT-N as the defining parameter with soil categorized in three classes [6]. Kim and Yoon [7] suggested classifying sites based on site period. Site classification based on V_s^{30} parameter was arrived at after studies on California data, but used in the several other regions irrespective of difference in local conditions. However, study in Anbazhagan et al. [8] showed that V_s^{30} is not appropriate in the region of

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shallow bedrock sites and hence, it may be meaningful to consider alternate site classification system instead of V_s^{30} for these regions. In the present study, shallow bedrock sites in Bengaluru, Chennai, Coimbatore and Vishakhapatnam (Vizag) have been analyzed. The SPT-N values are abundantly available for these regions and with appropriate correlations; shear modulus can be calculated [9].

Bolisetti et al. [10] used three industry-standard one-dimensional site response analysis programs to perform site response analysis for four representative soil sites ranging from hard rock to stiff soil. A comparison of the predictions of the different programs revealed that the industry-standard programs are in good agreement with each other for stiff sites subjected to low-intensity ground motions, but become increasingly different for analyses involving higher strains and at higher frequencies. The authors reported that the nonlinear programs result in close predictions of surface accelerations for all cases, despite employing different hysteresis rules, while the equivalent linear responses diverge from the nonlinear responses when the equivalent linear peak strains are greater than about 0.1% and are very different from the nonlinear responses at peak strains greater than 1%. For peak shear strains less than 0.1%, the computationally more efficient equivalent linear analysis can be used with high confidence. For peak shear strains greater than 0.1%, nonlinear methods should also be employed, with an increased weighting in the nonlinear methods with increasing peak shear strains. The differences between the equivalent linear and nonlinear responses are significant for periods less than 0.5 s for the soft sites, and less than 2 s for the stiff soil site. Similar recommendations are made in studies of [12, 13].

As the main objective of site response analysis is to evaluate the effect of local soil conditions on seismic waves, i.e., to find out “amplification” of seismic waves, various researchers have tried to account amplification by means of introducing different forms of amplification factor. Newmark and Hall [14] evaluated amplification factors for different damping ratios based on peak values of acceleration, velocity and displacement on a tripartite plot. On similar lines, Malhotra [15] proposed factors for acceleration, velocity and displacement zones with the introduction of a cutoff period separating high- and low-frequency zones. NEHRP [1] and IBC [2] suggested amplitude-dependent site amplification factors, F_a for a short period and F_v for a long period. Kim and Yoon [7] suggested similar short-period and long-period factors for Korean shallow profiles. [1, 2, 7] evaluated factors using ratio of response spectra (RRS) method given by Dobry et al. [16]. The short-period factor is evaluated using ratios between 0.1 and 0.5 s, while long-period factor is evaluated using ratios between 0.4 and 2.0 s.

The site response calculations are done using one-dimensional nonlinear approach in DEEPSOIL [11]. For

comparison with known data, the amplification factors are evaluated considering the period ranges similar to IBC [2]. Then, considering spectral signatures from response spectra for all site rock and surface motions, the factors are calculated for period range 0.01–0.06 s and 0.05–1.0 s. Based on the study only one mid-period amplification factor has been proposed. This amplification factor represents the significant amplification of the site. Dobry et al. [16] concluded that both short and long-period site coefficients are required for base accelerations greater than 0.2 g. In this study, the maximum base acceleration is 0.17 g. Hence, single amplification values represent amplification for a corresponding site group and it may be studied further.

Selection of Intraplate Earthquake

It is generally recognized that the selection of appropriate input ground motion is one of the main aspects that affect site response analysis. Several site response studies have been carried out in India, but most of them are in the absence of representative input ground motions. The selection of time histories includes records that closely match the site tectonic environment, controlling earthquake magnitudes and distances, local site conditions, response spectral characteristics, and, for geotechnical evaluations, duration of strong ground shaking. The shallow profiles under focus in this study are from Vishakhapatnam, Chennai, Coimbatore and Bengaluru. These locations are part of “Stable Continental Region (SCR)” and South India. There are significant damaging earthquakes in this “stable” region of Peninsular India (PI), e.g., Bhuj (2001, Mw 7.6), Koyna (1967, M 6.5), Latur (1993, Mw 6.1) and Jabalpur (1997, Mw 5.8). These were intraplate earthquakes and reported in previously unknown seismic activity region or unknown faults. These earthquakes also caused considerable damages close to the epicentre and far-away distance. However, recordings of these earthquake events are not available due to poor instrumentation, except Bhuj, an event which was recorded at structure. There are no acceleration time history data available in PI useful for site response and amplification estimation. Hence, intraplate recordings from around the world collected compiled by Anbazhagan et al. [17] to understand the duration of intraplate events has been considered for the site response study.

As per IS 1893 [6], cities, Bengaluru and Vishakhapatnam, fall in zone II, while Chennai and Coimbatore fall in zone III. Seismic intensity factor (S_a/g) for these zones is 0.10 and 0.16, respectively, according to Indian code. Hazard studies have been done by various researchers for different cities of peninsular India. Vipin et al. [18] gave contour maps for rock level Peak Horizontal Acceleration

(PHA) for peninsular India. Rock level PHA values for a 10% probability of exceedance in 50 years for Vishakhapatnam are 0.10 g, while for Chennai, Bengaluru and Coimbatore, it is around 0.16 g. Kumar et al. [19] carried out seismic hazard analysis of Visakhapatnam using probabilistic approach and gave the peak ground acceleration (PGA) as 0.114 g to 0.119 g for a 10% probability of exceedance in 50 years. Anbazhagan et al. [20] carried out probabilistic seismic hazard analysis of Bengaluru city and suggested PGA value of 0.121 g corresponding to 10% probability of exceedance in 50 years. For Chennai city, Ramanna and Dodagaodar [21] estimated peak ground acceleration for a 10% probability of exceedance in 50 years to be less than 0.1 g. Raghukanth [22] carried out a regression analysis of the frequency, magnitude distribution and suggested threshold magnitude for the cities as 4.0. Considering the PGA values from mentioned studies, it can be found that study area may have potential for PGA value of 0.05 g to 0.2 g. About 13 intraplate ground motion records available in this range of PGA values are identified and used in this study. Summary of selected ground motions is given in Table 1. Thirteen ground motions data are corrected by applying baseline correction and are further used for site response study. Acceleration time history plot of one of the selected motion (a1_enr) is shown in Fig. 1.

Selection of Shallow Bedrock Sites

The shear wave velocity obtained by MASW (Multi Channel Analysis of Surface Waves) surveys adjacent to corresponding bore hole locations at 51 locations are selected for site response analysis. Similar stiffness and thickness sites are eliminated. Shear modulus of soil

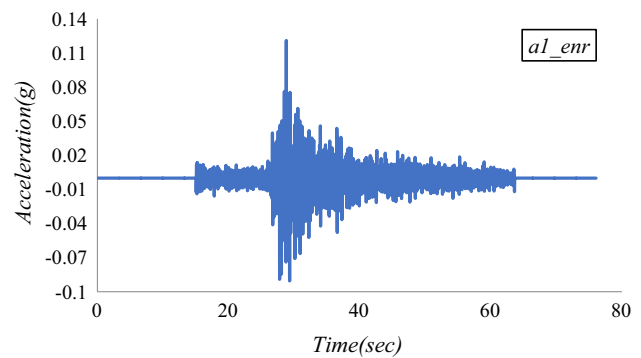


Fig. 1 One of the acceleration time histories used in study

column at each site has been estimated up to the input level by (a) previously identified G_{\max} versus SPT-N correlation if it is SPT profile and (b) by Eq. 1 if it is V_s profile. Variation of these shear modulus values with soil column depth is plotted in Fig. 2. It is noted from Fig. 2 that for few spots there is no data, so in order to fill the gap, SPT data from available bore logs are used to evaluate stiffness and same is superimposed on Fig. 2 (solid circle).

$$G = \rho V_s^2 \quad (1)$$

where ρ is density, G is shear modulus and V_s is shear velocity.

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \left(\frac{d_i}{N_i}\right)} \quad (2)$$

where \bar{N} is the equivalent N value, $\sum_{i=1}^n d_i$ is the summation of total depth, d_i and N_i denotes the thickness (in m) and standard penetration resistance, respectively. Variation of SPT N with respect to depth thus calculated is shown in Fig. 3.

Table 1 Summary of input motion selected for study

Earthquake	Station	Motion code	PGA (g)	Epi dist (km)	Magnitude (M_w)
Saguane 1988	St.-Ferreol	a1_enr	0.121	115.827	5.6
Saguane 1989	Quebec	a2_enr	0.0506	147.615	5.6
Saguane 1990	Baie-St-Paul	a7_ent	0.174	105.068	5.6
Saguane 1991	La	a8_enr	0.124	124.192	5.6
Saguane 1992	St.-Pascal	a9_en2	0.0558	164.996	5.6
Saguane 1993	Riviere-Ouelle	a10_ent	0.057	148.398	5.6
Saguane 1994	Chicoutimi	a16_ent	0.131	45.142	5.6
Saguane 1995	Andre-Lac-Jean	a17_enr	0.156	91.847	5.6
Saguane 1996	Les	a20_enr	0.126	112.94	5.6
Quebec 2005	A61-2005	A61-2005	0.07	29.696	5.4
Quebec 2006	A21-2005	A21-2005	0.084	13.3	5.4
Alaska 2008	A21-2008	A21-2008	0.077	5.553	4.7
Virginia 2011	VA	Virginia	0.098	53.5	5.8

Fig. 2 Stiffness variation with soil column thickness for selected profiles (color figure online)

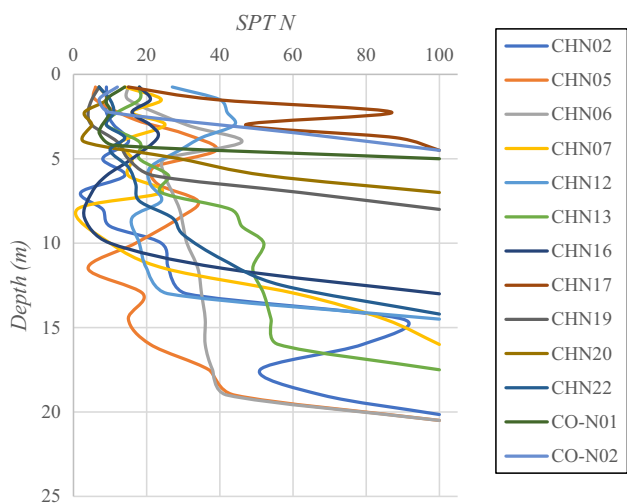
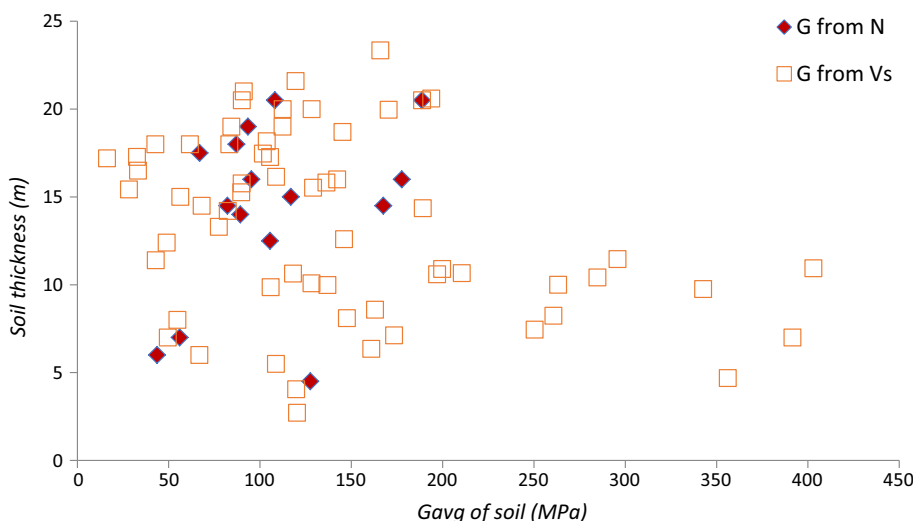
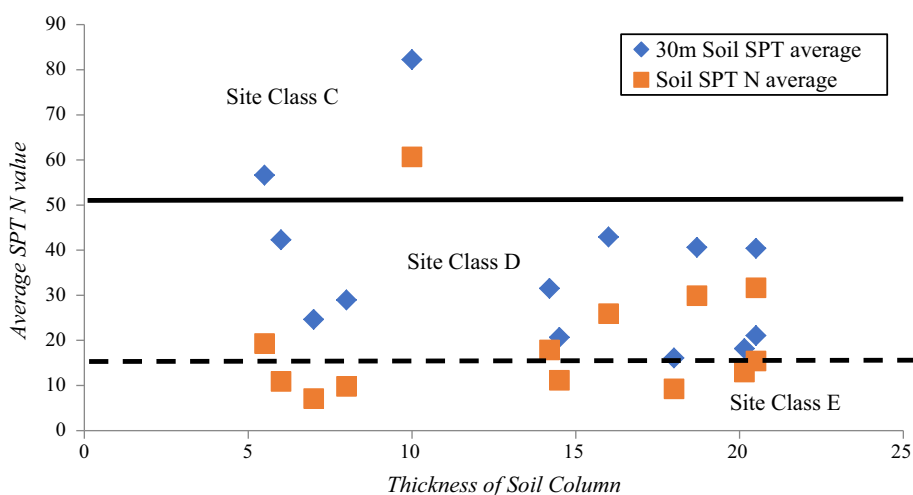


Fig. 3 Variation of SPT-N with depth for additional profiles selected

G_{max} values of each layer are used to estimate average G_{max} values up to input layer in each site. Variation of N with depth for the additional 13 profiles is shown in Fig. 3. Average N values up to soil thickness (orange circles) and N_{30} values based on borelog data (blue circles) for the profiles have been calculated according to Eq. 2 [1] and shown in Fig. 4. It can be observed that eight of thirteen profiles will have incorrect class representation if N_{30} is the criteria and NEHRP [1] classification is considered. Including rock values in soil thickness, i.e., N_{30} , has considerably altered average N values and site class as average N values are relatively lower than N_{30} values, due to exclusion of rock values.

The profiles are divided into five stiffness groups and given in Table 2. The characteristic site period T , which is dependent on both the shear wave velocity and layer thickness, is computed by Eq. 3 [7].

Fig. 4 Thickness of soil column and average SPT-N values for 30 m and soil thickness



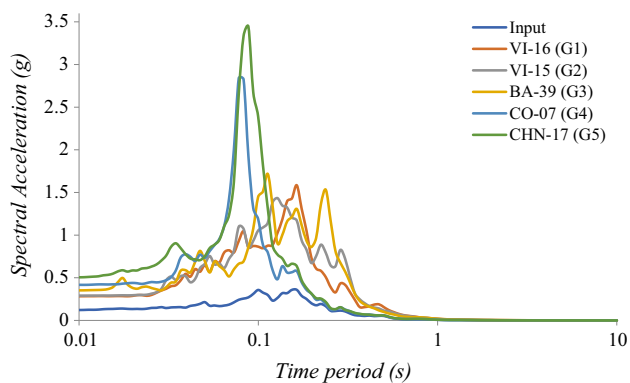


Fig. 5 Spectral result corresponding to different groups for “a1_enr” motion

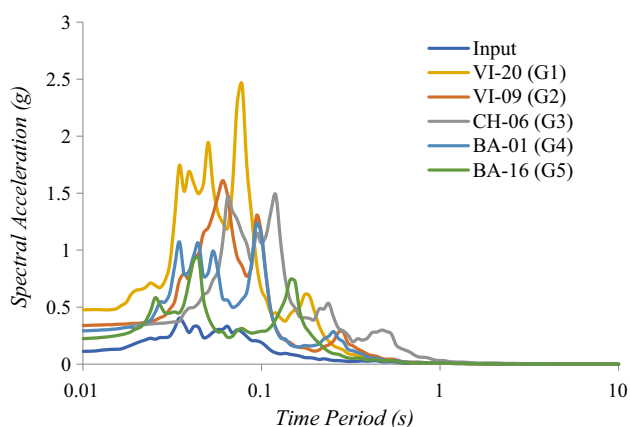


Fig. 6 Spectral response of all group for Virginia motion (color figure online)

$$T = 4 \sum_{i=1}^n \left(\frac{d_i}{V_{s_i}} \right) \quad (3)$$

where d_i is the thickness of each soil layer above the bedrock, V_{s_i} is the shear wave velocity of each soil layer and n is number of soil layers up to bedrock.

The notation VI-01 in table stands for Vizag V_s profile corresponding to first location. Similarly, notations CH, CO and BA stand for Chennai, Coimbatore and Bengaluru profiles, respectively. CHN and CON notations stand for SPT-N profiles in Chennai and Coimbatore. There are 8 profiles in class G1, 13 in G2, 21 in G3, 11 in G4 and 11 in G5. The natural site periods by Eq. 3 are in the range of 0.05 and 0.48 s. The average site periods for the five site class are 0.324, 0.265, 0.217, 0.175 and 0.117 s at G1, G2, G3, G4 and G5 sites, respectively. It is noted that with increase in stiffness of soil column, natural period decreases. This is expected as soil columns become more rigid with stiffness.

Site Specific Response Study

Fifty one shear wave velocity profile and 13 SPT-N profiles discussed in previous section has been used to estimate amplification of shallow bedrock sites. Parameters required for response calculation in DEEPSOIL [11] are layer thickness, density and shear velocity or shear modulus. Rest of the parameters are according to the G/G_{max} , and damping ratio curve is calculated by curve fitting procedure by the software. Rest of the input for analysis by software is done with input motion as within, rigid half space and damping ratio of 5%. Site response study has been carried out by giving input motion at layer which has V_s in range of 500 ± 100 m/s and $N = 100$ according to Parihar [23]. Shear modulus and damping curves are selected based on type of soil according to Anbazhagan and Parihar [24]. These 64 profiles and 13 input motions are used to understand amplification by carrying out nonlinear site response analysis by DEEPSOIL [11]. A total of 832 analyses are carried out on the 64 profiles and 13 motions. Spectral parameters and response spectrum has been evaluated for each analysis and which are further used for estimating amplification factor. Resulting spectral values

Table 2 Profile classification based on stiffness

Group	G_{avg} (MPa)	Profile	Natural period of soil column (s)
G1	0–50	VI-18, VI-17, VI-16, VI-22, VI-07, VI-20, VI-21, CHN-20	0.48, 0.36, 0.37, 0.38, 0.38, 0.24, 0.23, 0.15
G2	50–100	VI-09, VI-15, VI-04, CH-01, CH-03, VI-19, CHN-02, CHN-05, CHN-07, CHN-16, CHN-19, CHN-22, CO-N01	0.3, 0.33, 0.23, 0.25, 0.27, 0.28, 0.34, 0.35, 0.29, 0.28, 0.17, 0.25, 0.11
G3	100–150	BA-39, VI-11, CH-06, CH-04, BA1-09, BA1-18, BA1-17, CH-09, VI-05, CO-12, CO-13, CH-07, VI-13, BA-22, CH-02, BA-11, VI-06, VI-01, CHN-12, CHN-13, CO-N02	0.26, 0.26, 0.27, 0.15, 0.26, 0.3, 0.31, 0.27, 0.27, 0.14, 0.17, 0.14, 0.32, 0.24, 0.22, 0.14, 0.17, 0.11, 0.22, 0.25, 0.08
G4	150–250	CH-05, CO-07, VI-03, BA-01, CO-10, CH-08, BA-49, VI-02, VI-12, BA-32, CHN-07	0.08, 0.1, 0.29, 0.23, 0.09, 0.29, 0.24, 0.13, 0.12, 0.18, 0.26
G5	250–500	CO-03, CO-20, BA-17, CO-02, BA-16, CO-09, BA-13, CO-01, CHN-17, CO-04, BA-47	0.09, 0.09, 0.36, 0.12, 0.12, 0.05, 0.06, 0.1, 0.1, 0.09, 0.08

are clubbed together according to the site group based on the stiffness.

Results Based on Modulus Only

Typical results belonging to each group from the 832 analyses are presented here. Amplification values estimated based on peak ground acceleration (PGA), i.e., Spectral acceleration at zero period and peak spectral acceleration (PSA) of input and surface values. Vizag profile VI-16 is a clay profile with top 13 m pure clay overlying rock. Plasticity index (PI) for clay ranges between 17 and 31. Shear modulus of the soil column analyzed is 32.69 MPa and hence falls in group G1. Natural period of the column as calculated from Eq. 2 is 0.37 s. Result for the profile analyzed for “a1_enr” input motion is presented in Fig. 5. It can be observed that for period 0.01–0.02 s, the amplification is a constant value of about 2.32. Beyond this period, there is significant amplification till 0.5 s with ratio of peak spectral values being 4.42. Peak of input and surface is reached at same period of 0.16 s.

Vizag profile VI-15 is a sandy clay profile with top 11.6 m of clay overlying rock. Plasticity index for clay ranges between 17 and 31. Shear modulus of the soil column analyzed is 61.61 MPa and hence falls in group G2. Natural period of the column as calculated from Eq. 2 is 0.33 s. Result for the profile analyzed for “a1_enr” input motion is presented in Fig. 5. It can be observed that from period 0.01 to 0.025 s, the amplification is a constant value of about 2.39. Beyond this period, there is significant amplification till 0.4 s with ratio of peak spectral values being 4.

Bengaluru profile BA-39 is a silty sand profile with top 11.18 m of sand and silty sand overlying rock. Shear modulus of the soil column analyzed is 101.59 MPa and

hence falls in group G3. Natural period of the column as calculated from Eq. 2 is 0.26 s. Result for the profile analyzed for “a1_enr” input motion is presented in Fig. 5. It can be observed that from period 0.01 to 0.03 s, the amplification is a constant value of about 2.89. Beyond this period, there is significant amplification till 0.5 s with ratio of peak spectral values being 4.79.

Coimbatore profile CO-07 is a mix profiles with 2-m-thick silty clay and gravel overlying rock. Shear modulus of the soil column analyzed is 162.98 MPa and hence falls in group G4. Natural period of the column as calculated from Eq. 2 is 0.1 s. Result for the profile analyzed for “a1_enr” input motion is presented in Fig. 5. It can be observed that from period 0.01–0.03 s, the amplification is a constant value of about 3.42. Beyond this period, there is significant amplification till 0.2 s with ratio of peak spectral values being 7.92.

Chennai profile CHN-17 is a mix profile consisting of 10-m-thick silty sand, sandy clay and sand overlying rock. Plasticity index for the clay is 29. Shear modulus of the soil column analyzed is 263.21 MPa and hence falls in group G5. Natural period of the column as calculated from Eq. 2 is 0.1 s. Result for the profile analyzed for “a1_enr” input motion is presented in Fig. 5. It can be observed that from period 0.01–0.02 s, the amplification is a constant value of about 4.15. Beyond this period, there is significant amplification till 0.2 s with ratio of peak spectral values being 9.61.

Results Based on Modulus Calculated from V_s

Results from analysis of V_s profiles, corresponding to all proposed groups, are presented in Fig. 8. Site group G1 profile VI-20 (Table 3) is having 8.2 m of clay over rock with plasticity (PI) index varying from 22 to 57. Site group

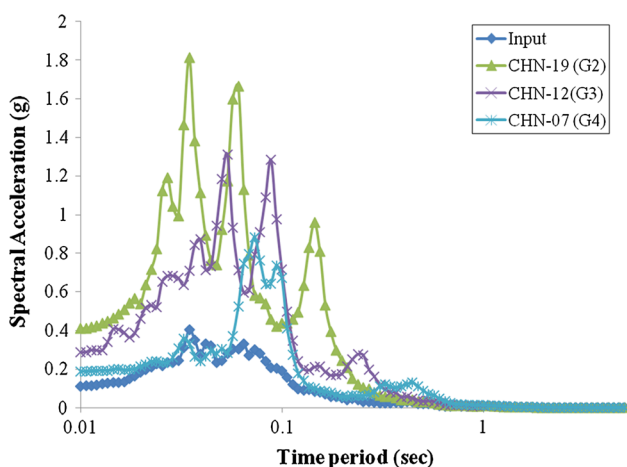


Fig. 7 Spectral response of SPT-N profiles in different groups for Virginia motion

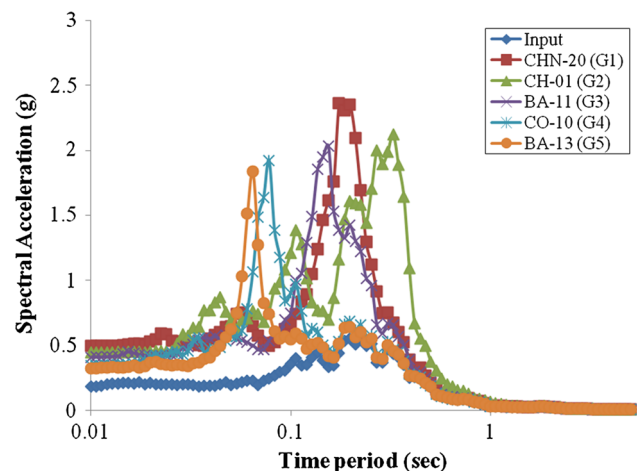


Fig. 8 Typical spectral response different stiffness group profiles corresponding to “a7_ent” motion

Table 3 Input for profile VI-20

Layer no.	Soil type	Thickness (m)	Unit weight (kN/m ³)	Shear velocity (m/s)	Damping ratio (%)	Ref. strain (%)	Ref. stress (MPa)	β	s	b	d	$P1$	$P2$	$P3$
1	Silty clay	0.55	11.93	106.66	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
2	Silty clay	0.55	11.93	106.66	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
3	Silty clay	0.862	12.03	144.172	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
4	Silty clay	1.288	12.03	144.72	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
5	Silty clay	1.075	15.824	144.72	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
6	Sandy clay	1.075	15.824	144.72	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
7	Silty clay	1.557	16.95	212.87	0.9858	0.3208	0.18	1.2	0.885	0	0	0.8	0.398	0.95
8	Silty clay	1.243	16.95	246.944	0.9858	0.3208	0.18	1.2	0.885	0	0	0.8	0.398	0.95
9	W rock	3.2	24.525	435.336	2.9782	0.1452	0.18	1.455	0.99	0	0	1	0.4	3.05

β , s , $P1$ and $P2$ are unitless curve fitting parameters, while d is small strain damping

G2 profile VI-09 is having 10.7 m of clay with PI of 28, overlying rock. Site group G3 profile CH-06 is having 7.143 m of sand over rock. Site group G4 profile BA-01 is having 12.022 m of sand overlying rock. Site group G5 profile BA-16 01 is having 6.451 m of sand overlying rock.

It is observed that for lower stiffness site classes, amplification is more, i.e., as we move from G5 to G1, amplification increases. Amplification factor with respect to PGA is 4.253 (G1), 3.024 (G2), 3.05 (G3), 2.513 (G4) and 2 (G5). Group G1 has higher spectral values compared to rest of site classes, and the values reduce with increasing stiffness. This trend follows on expected lines as amplification is supposed to increase with reduction in stiffness. This is clearly visible for short time periods ($T < 0.1$ s) though similar observation cannot be seen for larger time periods.

Results Based on Modulus Calculated from SPT-N Value

Results corresponding to available groups based on SPT-N profile analysis are presented in Fig. 7. Group G2 profile CHN-19 (Table 4) is having 7 m mix of silty clay with plasticity (PI) index varying from 20 to 35 and clayey sand, overlying rock. Group G3 profile CHN-12 is having 14.5 m of mostly sand over rock. For the 1.5-m-thick clay, layer in profile PI = 10 is assumed. Group G4 profile CHN-07 is having 14.5 m of sand and clay overlying rock. PI for clay is 54.

It is observed that for similar to the V_s profiles, amplification is more for lower stiffness site group in N profiles as well. Amplification factor with respect to PGA is 3.678 (G2), 2.559 (G3) and 1.676 (G4).

It can be observed from the results that spectral peaks are occurring toward time period less than 1 s. This has

been observed for all the results corresponding to all the intraplate earthquakes used in this study.

Amplification Factors

Amplification of ground motion at various portions of the spectrum is conveniently expressed as amplification factor or site coefficient. Newmark and Hall [14] suggested amplification factors corresponding to acceleration, velocity and displacement response for different damping values, calculated from acceleration time history. They used the ratios between spectral values and peak ground motion parameters (PGA, PGV and PGD) to determine the average amplification factors used in the construction of smooth-response spectra. Hall et al. [25] assumed that the SAs for periods < 0.33 s are sensitive to PGA, those for periods between 0.33 and 3.33 s are sensitive to PGV, and the rest is sensitive to PGD. Malhotra [15] also suggested amplification factors corresponding to acceleration, velocity and displacement response for different damping values, calculated from acceleration, velocity and displacement time histories. IBC [2] suggested the site coefficients as a short-period amplification factor (corresponding to 0.2 s time period) F_a and long-period amplification factor (corresponding to 1 s time period) F_v . The coefficients were calculated using the ratio of response spectra (RRS) or ratio of Fourier spectra (RFS) of the soil and corresponding rock records. The coefficients are provided as a function of site class and mapped spectral acceleration, where site class is determined according to the shear velocity, SPT-N or undrained shear strength. For IS 1893 [6], the spectral acceleration coefficient is capped at 2.5 for different soil types (classified based on SPT-N value). According to Dobry et al. [16], the site coefficients F_a specified in the

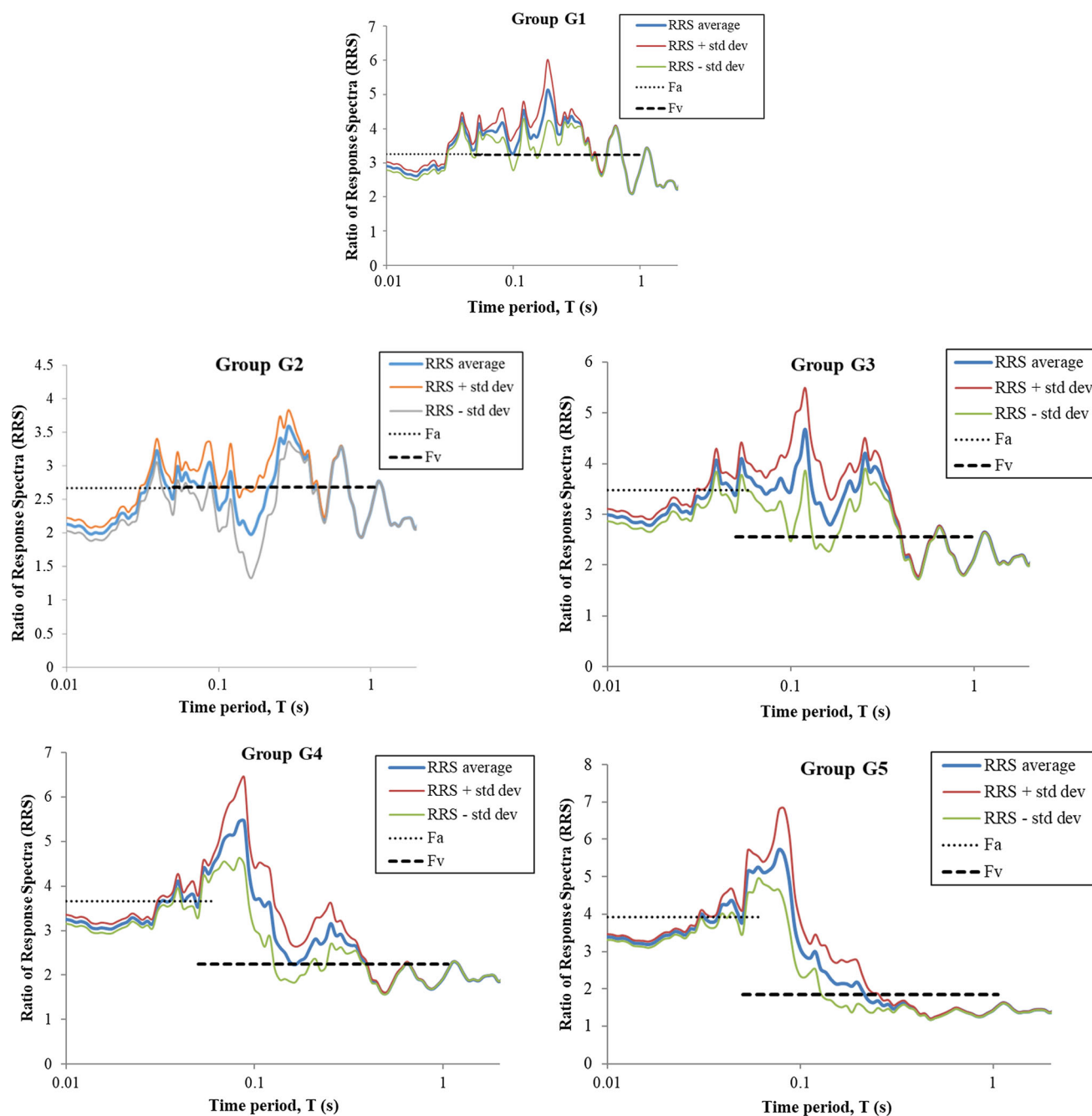


Fig. 9 Amplification factors evaluated in this study for motion “a1_enr,” PGA 0.12 g for all five groups

IBC [2] provisions are about the average value and the site coefficients F_v are approximately the average + 1σ values. In IBC [2] provisions, F_a is determined as the average value of RRS over the short-period band 0.1–0.5 s and F_v is determined as the average + 1σ of RRS over the long-period band 0.4–2.0 s [16].

In the present study, the site coefficients were divided into three types of coefficients as short-period (0.01–0.1 s) amplification factor, F_1 , mid-period (0.1–0.5 s) amplification factor, F_2 and long-period (0.4–2 s) amplification

factor, F_3 . Factors F_1 , F_2 and F_3 using a ratio of response spectra are calculated using Eqs. 4, 5 and 6 [16].

$$F_1(RRS) = \frac{R_{soil}}{R_{rock}} \frac{1}{0.09} \int_{0.01}^{0.1} \frac{RS_{soil}(T)}{RS_{rock}(T)} dT \tag{4}$$

$$F_2(RRS) = \frac{R_{soil}}{R_{rock}} \frac{1}{0.4} \int_{0.1}^{0.5} \frac{RS_{soil}(T)}{RS_{rock}(T)} dT \tag{5}$$

Table 4 Input for profile CHN-19

Layer no.	Soil type	Thickness (m)	Unit weight (kN/m ³)	Shear modulus (kPa)	Damping ratio (%)	Ref. strain (%)	Ref. stress (MPa)	β	s	b	d	$P1$	$P2$	$P3$
1	Silty clay	0.75	15.68	39,966.12	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
2	Silty clay	0.75	15.68	32,440.88	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
3	Silty clay	0.75	15.68	28,249.34	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
4	Silty clay	0.75	15.68	32,440.88	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
5	Silty clay	1	15.68	55,824.33	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
6	Silty clay	1	15.68	64,107.34	0.9615	0.209	0.18	1.47	0.84	0	0	0.896	0.4	0.78
7	Clayey silty sand	1	17.64	81,042.11	0.2084	0.0938	0.18	1.56	0.945	0	0	0.678	0.194	0.45
8	Sandy silty clay	1	17.64	152,630.7	0.9858	0.3208	0.18	1.2	0.885	0	0	0.8	0.398	0.95
9	W rock	1	24	468,572.1	2.9782	0.1452	0.18	1.455	0.99	0	0	1	0.4	3.05

$$F_3(\text{RRS}) = \frac{R_{\text{soil}}}{R_{\text{rock}}} \frac{1}{1.6} \int_{0.4}^2 \frac{\text{RS}_{\text{soil}}(T)}{\text{RS}_{\text{rock}}(T)} dT \quad (6)$$

where RS_{soil} and RS_{rock} are response spectra on soil and rock at a given period T , and R_{soil} and R_{rock} are the hypocentral distances of soil and rock stations. The ratio of $R_{\text{soil}}/R_{\text{rock}}$ was assumed to be 1.0 in this study as rock and surface spectral hypocentral distance is similar.

Typical spectral acceleration plot of different site class for motion of Factors F_1 , F_2 and F_3 are calculated for all 13 motions and averaged. These are presented according to different stiffness site group in Table 5. IBC [2] factors mentioned are corresponding to the mapped spectral acceleration of less than equal to 0.25 for short-period factor and less than equal to 0.1 for long-period factor. This is because these values are highest among the recommendations, and results from our study have factors at high end as well. Kim and Yoon [7] evaluated response of 162 sites in Korean peninsula using three outcrop motions. As part of the study, they evaluated amplification factors F_a and F_v , similar to NEHRP [1]. Same are mentioned in Table 5 for

comparison purpose. The values correspond to earthquake motion of 0.11 g PGA.

It is observed that factors F_2 and F_3 constantly decrease with increase in stiffness with exception of F_2 having a slight increase from G2 to G3. A similar trend is noted in IBC [2] factors as site coefficients decrease with increasing site class. It is to be considered that for IBC [2], site class A ($V_s > 1500$ m/s) represents the highest stiffness, while for our study, sites group G5 ($G > 250$ MPa) is of the highest stiffness value. For F_1 , however, the values increase with stiffness with exception of G2. The maximum factor noted is 4.44 and is for least stiff sites group G1 corresponding to mid-period, i.e., 0.1–0.5 s. This is understood to be so because the average natural period of sites in this site class is 0.324 s; hence, maximum amplification is expected to be around this time period value. IBC [2] has maximum coefficient value (3.5) corresponding to the long period for site class E. The difference is understood to be because of different geology of the sites considered for evaluation of coefficients. It can also be noticed from natural periods of different site class that with increasing stiffness, the spectra are shifting toward lower values; hence, F_1 is increasing

Table 5 Comparison of amplification factors and site coefficients

Site Class	Our study/IBC		From this study				[2]		[7]		From this study		[2]		[7]	
	F_1	σ^a	F_2	Σ	F_a	F_a	Ave ^b	σ	F_3^c	F_v	F_v	F_v	F_v			
G1/E	3.468	0.820	4.440	0.985	2.500	1.520	2.536	0.883	3.419	3.500	2.540					
G2/D	2.840	0.539	3.277	0.559	1.600	2.090	2.129	0.431	2.560	1.700	1.610					
G3/C	3.654	0.567	3.392	0.389	1.200	1.690	1.833	0.348	2.180	1.400	1.170					
G4/B	4.259	0.539	2.752	0.333	1.000	1.090	1.648	0.290	1.938	1.000	1.010					
G5/A	4.295	0.496	2.160	0.312	0.800		1.377	0.157	1.534	0.800						

^a σ means the standard deviation of factors by Eqs. 4, 5, 6

^bAve means arithmetic average of RRS in the interval 0.4–2.0 s

^c F_3 means sum of Ave and 1 standard deviation

Table 6 Amplification factors based on acceleration, velocity and displacement

Author	Factor notation		
	A	V	D
[14]	2.71	2.3	2.01
[15]	α_A	α_V	α_D
	2.12	1.65	1.39

A, V, D means acceleration, velocity and displacement, respectively

instead of decreasing. Amplification factors suggested in studies of Malhotra [15] and Newmark and Hall [14] (refer Table 6) are lower than values in present study. No information regards stiffness is available for these studies in the mentioned literature; hence, it is difficult to ascertain if the difference is due to different geological conditions. However, it can be noted that the amplification factor for acceleration dominated (high frequency) region is higher for both the studies, while in our case that is true only for site group G3, G4 and G5. Studies from [2] and [7] are similar except for site class S_D and S_E . Though the values are corresponding to site classes from V_s , it is difficult to ascertain the shear modulus value for the same. Still, as IBC [2] class A and B indicate high stiffness and compared coefficients for these classes with site group G5 which is the stiffest in our case. Trend of amplification factors obtained from the present response study is comparable with the previous study; however, these values are different from site coefficients given in the code and previous study. Detailed analysis required to find site coefficients of the study area which may be carried in the future. To further understand the change in amplification with stiffness, plot of spectral acceleration response corresponding to “a7_ent” motion is presented in Fig. 8. This should be seen in continuation with Figs. 5, 6 and 7. It can be observed from the figures that the PGA and PSA values gradually decrease with increasing stiffness, i.e., PGA and PSA decrease as the average G of soil column increases. This holds true irrespective of soil type as CHN-20 is a mix of sand and clay, CH-01 is pure sand, BA-11 is predominantly clay with sand trace, CO-10 is gravelly sand, while BA-13 is pure sand. It can also be observed that in general, occurrence of peak spectral value shifts leftwards with increasing stiffness.

Results and Discussion

It is noticed from the spectral signatures of surface response that most of amplification occurs before 1 s period. This can be noticed from typical plots in Figs. 5, 6 and

7. Similar observations are for almost all the spectral responses obtained from the 832 analyses. Amplification factors calculated as ratios of PGA and PSA might be misleading as there is significant amplification around peak value as well. It is also noticed that from period range 0.01–0.06 s, amplification remains nearly constant, while peak values are reached in period range 0.08–0.8 s periods. Hence, it may be more reasonable to evaluate amplification factor in the period range of 0.01–1.0 s for this region.

Calculation of Final Amplification Factors

Two factors are calculated from analyses results: constant amplification factor, F_C , corresponding to period range 0.01–0.06 s and significant amplification factor, F_S , corresponding to period range 0.05–1.0 s. For a particular site class, at each time period, lognormal median of spectral values (RS_{soil}) is calculated according to Eq. 7. Ratio of this median surface acceleration to rock spectral acceleration (RS_{rock}) corresponding to each site period is calculated. This ratio is $RS_{soil}(T)/RS_{rock}(T)$ and is to be used in Eqs. 8 and 9, for the corresponding period. R_{soil}/R_{rock} is assumed to be 1.0 similar to previous section. Then, amplification factors F_C and F_S are calculated as according to Eqs. 7 and 8. The calculation is carried out for all site classes. Values obtained thus are presented in Table 7.

$$RS_{soil} = \exp\left(\frac{1}{N} \cdot \sum_{i=1}^N \log RS_{soil_i}\right) \tag{7}$$

$$F_C(RRS) = \frac{R_{soil}}{R_{rock}} \frac{1}{0.05} \int_{0.01}^{0.06} \frac{RS_{soil}(T)}{RS_{rock}(T)} dT \tag{8}$$

$$F_S(RRS) = \frac{R_{soil}}{R_{rock}} \frac{1}{0.95} \int_{0.05}^{1.0} \frac{RS_{soil}(T)}{RS_{rock}(T)} dT \tag{9}$$

where N is the number of profiles in site class for which calculation is made.

The results are further divided according to the input motion PGA values and arithmetic average calculated. The results of these calculations are presented in Table 8. It is concluded that the factors do not sway much from each other for these PGA ranges.

Trends of Amplification Factors

The amplification factor evaluated is an arithmetic average corresponding to all 13 motions and presented in Table 9. Standard deviations for each of the factors are also mentioned to get an idea of variation among factors belonging to different soil profiles and motions. From this study, it has been observed that in general, amplification factor is

Table 7 Amplification factors calculated for 13 ground motions for the five stiffness groups from this study

Motion	Site class	Factor	
		F_C	F_S
a1-enr	G1	3.671	3.568
	G2	2.758	2.811
	G3	3.648	2.801
	G4	3.927	2.429
	G5	4.329	1.790
a2-enr	G1	5.078	3.134
	G2	4.061	2.617
	G3	4.729	2.442
	G4	5.006	2.196
	G5	5.112	1.695
a7-ent	G1	3.125	2.553
	G2	2.310	2.102
	G3	3.150	2.126
	G4	3.184	1.959
	G5	3.128	1.595
a8-enr	G1	3.323	2.603
	G2	2.570	2.219
	G3	3.440	2.247
	G4	3.495	1.947
	G5	3.956	1.654
a9-en2	G1	5.290	3.599
	G2	3.773	2.638
	G3	4.834	2.490
	G4	4.423	2.133
	G5	4.795	1.733
a10-ent	G1	3.478	2.838
	G2	3.703	2.200
	G3	4.302	2.155
	G4	4.183	1.850
	G5	4.264	1.633
a16-ent	G1	2.163	3.879
	G2	2.040	2.641
	G3	2.815	2.431
	G4	3.175	2.173
	G5	3.561	1.837
a17-enr	G1	1.858	5.850
	G2	1.922	3.079
	G3	2.647	2.759
	G4	3.070	2.560
	G5	3.165	2.061
a20-enr	G1	2.342	3.581
	G2	2.278	2.766
	G3	3.478	2.724
	G4	3.634	2.148
	G5	4.214	1.821
A21-2005	G1	3.319	3.768

Table 7

A21-2008	G2	2.435	2.823
	G3	3.105	2.456
	G4	3.559	2.276
	G5	3.737	1.927
	G1	2.730	4.475
A61-2005	G2	1.908	3.024
	G3	2.680	3.231
	G4	2.688	2.588
	G5	2.984	2.074
	G1	2.911	4.634
virginia	G2	2.302	3.224
	G3	3.091	2.798
	G4	3.462	2.471
	G5	3.702	2.036
	G1	3.050	3.484
	G2	2.689	2.690
	G3	3.325	2.633
	G4	3.760	2.468
	G5	4.143	2.118

Table 8 Amplification factors for differing PGA ranges

Site class	PGA (g)	Factor	
		F_C	F_S
G1	0.05–0.1	3.694	3.705
		2.982	2.745
		3.724	2.601
		3.869	2.283
		4.105	1.888
G1	0.1–0.16	2.747	3.672
		2.313	2.603
		3.196	2.515
		3.414	2.203
		3.726	1.793

Table 9 Amplification factors calculated from this study

Site class	F_C	Σ	F_S	σ
G1	3.257	1.005	3.24	0.657
G2	2.673	0.721	2.68	0.342
G3	3.48	0.725	2.561	0.307
G4	3.659	0.618	2.246	0.243
G5	3.93	0.639	1.844	0.183

σ is the standard deviation

constant for all site classes in the period range of 0.01–0.06 s. The factors vary from 2.67 to 3.93 for sites group G2 factor being the lowest. This trend is in disagreement with the general understanding that at very high frequencies or at very low periods, spectral accelerations converge toward peak ground acceleration. That is expected in theory as a low period indicates stiff soil column and

that stiffer a column is, the more is it likely to behave close to input vibration. Constant nonzero amplification value in the present study seems to indicate that even in stiff condition, there is an amplification. Values of standard deviation for amplification are also on the higher side.

Amplification factor of 0.05 to 1.0 s is decreasing consistently from site group G1 to G5. The trend is more evident as compared to other factors discussed earlier. It can be noted here that period range considered to calculate the amplification is based on input and surface spectral signature, which is different from the period range given in IBC [2]. Amplification value trend is in coherence with understanding that with increasing stiffness, energy loss from bottom of soil column to top is less; hence, amplification is reduced as well.

To highlight variation of ratio of response spectra (RRS) with time period (T), $RS_{\text{soil}}(T)/RS_{\text{rock}}(T)$ values are plotted with T in Fig. 9 for motion with PGA 0.12 g. $RRS \pm$ standard deviation curves are also plotted to visualize variation. Final amplification factors, which are evaluated from these RRS values through Eqs. 7 and 8, are plotted as well to give an idea regards their location with respect to the lognormal median of spectral ratios.

Conclusions

Different codal provisions accepted in practice in different countries have grouping based on SPT-N, V_s and/or time period. The present study is very important as it suggests that grouping be done based on shear modulus as it takes into account the stiffness. In this study, first intraplate motions from seismically stable regions are collected for the response evaluations of the study area. Intraplate ground motions selected have PGA in the range of 0.05–0.17 g. Range of PGA values of the collected motions is well with the recommendations made by several researchers for peninsular India and zone intensity factor by IS 1893 [6]. The shallow site profiles collected from MASW and SPT tests conducted at various locations in PI are selected. Site response analysis of 64 profiles is carried out by giving input of 13 ground motion by adopting the nonlinear program of DEEPSOIL [11]. Input for each site is given at layer having a V_s value of more than 500 ± 100 m/s and shear modulus and damping curves are selected based on soil type. Surface response parameters are arrived and used for estimating amplification factor for the study region. Amplification factors are calculated according to RRS method and compared with other reported values. Amplification factors are recalculated for different range time period based on the spectral signatures of all response results. The following observations have

Table 10 Amplification factors proposed from this study

Site class	F_s
G1 ($G < 50$ MPa)	3.24
G2 ($50 \text{ MPa} < G < 100$ MPa)	2.68
G3 ($100 \text{ MPa} < G < 150$ MPa)	2.561
G4 ($150 \text{ MPa} < G < 250$ MPa)	2.246
G5 ($G > 250$ MPa)	1.844

been made while evaluating site response and amplification factors in this study.

1. Acceleration spectra show similar trends for different site classes irrespective of the fact that profiles are selected based on V_s values or SPT-N values. This is observed over a wide range of results and indicates shear modulus as an influencing parameter.
2. Comparison of spectral signatures for different site class suggests that amplification reduces as the modulus of the soil column increases. Thus, it may be appropriate to classify sites based on shear modulus of soil column.
3. Amplification for all the shallow profiles and recorded intraplate earthquake is occurring in short-period range, i.e., period less than 1 s.
4. The trend of amplification factors calculated from the RRS method for periods 0.01–0.1 s, 0.1–0.5 s and 0.4–2.0 s is comparable with [2, 7] site coefficients.
5. Amplification factors are recalculated for periods 0.01–0.06 s and 0.05–1.0 s, which are decided by spectral signatures for profiles. Trend of amplification factors estimated are comparable with previous study.
6. Summary of the site specific amplification factors estimated in this study is presented in Table 10. Shear modulus can be the defining parameter for seismic characterization of shallow bedrock sites.

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