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ORIGINAL PAPER

Impact of Higher Frequency Content of Input Motion Upon Equivalent Linear Site Response Analysis for the Study Area of Delhi

Joy K. Mondal · Abhishek Kumar

Abstract While travelling through the subsoil layers, earthquake generated bedrock motions get modified significantly due to local soil and should be quantified using ground response analysis. Present study concentrates on equivalent linear method of site response analysis in SHAKE2000 software. It is a frequency based analysis tool having default frequency set to 15 Hz. While due consideration is given to amplitude, no to very limited information about the frequency content of the input motion to be considered in ground response analysis is available. In the present work, the effect of the maximum frequency of ground motion in site response analysis using SHAKE2000 is examined. Two sets of analyses are carried out in this work based on 30 globally recorded input motions. In the first analyses, input motion up to 15 Hz maximum frequency, which is a default value in SHAKE2000 is considered while second analyses are based on considering each of the 30 input motions up to the Nyquist frequency. Comparing the results from the two sets of analyses highlight that selection of maximum frequency in SHAKE2000 has considerable effect in ground motion amplification at different depths. As a result, even the peak ground acceleration which controls the building behavior and damage scenario, is going to change considerably even in case same input motion is used in the analysis.

Keywords Local site effect · Equivalent linear analysis · Nyquist frequency · PGA

1 Introduction

Earthquakes cannot be prevented or predicted. Earthquake hazards are catastrophic in nature. If the probable damages related to earthquake can be predicted, earthquake hazards can be minimized. An earthquake wave gets modified in its properties while travelling through the soil to the surface. The presence of subsoil between the bedrock and the surface alters the ground motion generated during an earthquake (EQ) at the bedrock level once it reaches the surface. This phenomenon is known as the local site effect and has considerable influence in changing the ground motion characteristics namely amplitude, frequency content and duration of motion between the bedrock and the surface (Kumar et al. 2015, 2016; Anbazhagan et al. 2010). Researchers are more interested in quantifying these modified waves at the surface where the actual damage scenario during an EQ is felt. Depending upon the subsoil characteristics, the amount of damage during an EQ may vary from

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epicentral region to distant locations as reported worldwide. Classical examples where the presence of local soil enhanced the damages include the 1985 Michoacan EQ. During this EQ, ground motions between the bedrock and the surface were amplified by five times resulting in significant damages in the city of Mexico located about 600 km away from the epicenter. Further, the effects of local soil were evidenced during the 1989 Loma Prieta EQ. Another example where considerable damages were reported away from the epicenter includes the 1999 Chamoli EQ (M_w-6.8). Even though the epicenter for this event was located between the lesser and the higher Himalayas, it caused number of building damages in Delhi and Dehradun located beyond 200 km from the epicenter. Ground shaking due to this event was felt up to Nepal in the east, Haryana in the north, Pune in the south-west and had triggered numerous landslides (Jain et al. 1999; Mahajan and Virdi 2001; Kumar et al. 2015, 2016). In the year 2001, the western extent of Gujarat was shaken severely by Bhuj EQ (M_w-7.7). This event triggered large scale ground failures and liquefaction in locations of Lodai and Umedpur located about 50 km from the epicenter. Sand blows were also triggered in Chobari regions about 100 km from the epicenter (Rajendran et al. 2001). Significant soil amplification was observed in Ahmedabad located 350 km from the epicenter resulting in considerable damages to taller buildings (Narayan and Sharma 2004). During 1989 Loma Prieta EQ, the ground motions were amplified by 2–4 times at soft soil sites compared to the rock sites in the San Francisco-Oakland region which was located about 120 km away from the epicenter, thus causing tremendous damage (Housner 1989 as per Finn and Wightman 2003). On 7th April 2011, the country of Japan was hit by a great EQ (M_w-9.0). It was named as Sendai EQ since the epicenter was located 130 km east coast of Sendai in the Pacific Ocean. This was the biggest EQ ever recorded in Japan. As a result, large amount of liquefaction and uneven settlements were evidenced in the city of Maihama and Tokai Mura which were located beyond 150 km from the epicenter (Nihon 2011). Sikkim EQ (M_w-6.8) is another classical example of local site effect. Even though it was a moderate size EQ, it had triggered numerous landslides as well as building damages in the regions of Mangan, Jorethang, lower Zongue, Chungthang etc.

Above discussions are the clear evidences from India and worldwide where considerable damage seven at larger distances were observed during a moderate to great EQ due to local site effects. An effective way to predict these surface motions is site response analysis. There are three numerical ways in which a site response analysis can be conducted namely; linear, equivalent linear and nonlinear. Among these three methodologies, the equivalent linear method is the mostly followed because of its simplicity and reasonable accuracy. SHAKE (Schnabel et al. 1972) was the first program developed based on equivalent linear approach to solve ground shaking problem in frequency domain. Inherent soil nonlinearity was taken into account by employing the concept of complex secant shear modulus. Since then, many computer programs have been developed on different platforms to perform equivalent linear site response analysis. Sugito et al. (1994) developed FDEL considering frequency dependent soil properties into account. Further, Berdet et al. (2000) developed EERA, which can perform site response analysis in equivalent linear method using visual basic platform. At present, SHAKE2000 which is an improved and user-friendly version of SHAKE (Schnabel et al. 1972), is a widely accepted software for the equivalent linear site response analysis. Typical input require for a site response analysis in SHAKE2000 includes input motion and dynamic soil properties. In the absence of regional ground motion records, input motions are selected from other regional records corresponding to similar seismic activity or generated synthetically using regional parameters. Kumar et al. (2016) highlighted that the amplitude of input motion controls the amplification in the ground motion between the bedrock and the surface. In the absence of regional ground motion records thus, performing ground response analysis based on one or two ground motions from other regions may not be appropriate. Further, outcomes of such site response analysis will have limited applicable in quantifying induced effects such as liquefaction and landslides as well as will not provide proper assessment of damage scenario during future EQ. Thus, wide range of input motions covering considerable variation in ground motion characteristics should be considered as highlighted by Kumar et al. (2016). While due importance is given to amplitude of input ground motion and dynamic soil

properties, frequency content of ground motion which to be considered for site response analysis in SHAKE2000 has not been explored that much significantly. It has to be highlighted here that SHAKE2000 is a frequency domain based analysis, enough attention should be paid to the frequency content of the input motion as well. At present very limited literature exists highlighting the role of higher frequency content on input motion upon ground response analysis. Kumar (2012), attempted to understand the response of Lucknow soil columns considering 18 typical regional ground motions records covering wide range of ground motion parameters. Based on the work, Kumar (2012) reported maximum amplification of 6 iin the north and northwestern parts of Lucknow. Choudhury et al. (2015) performed equivalent and nonlinear ground response analyses for borehole locations in Mumbai while are prone to liquefaction potential. Ground motions recorded during 1995 Kobe, 2001 Bhuj EQ and 1989 Loma Prieta EQ were used as input motion. As per Choudhury et al. (2015), ground response analysis using 2001 Bhuj EQ showed maximum amplification due to higher frequency content as well as duration in comparison to other ground motion records causing more cyclic loading of the soil column in comparison to other motions. In another work, while assessing the soil response for Jawaharlal Nehru Port (JNPT) and Mumbai port, Desai and Choudhury (2015a) also found considerable amplification in higher frequency range of 36–45 Hz. Similarly, while understanding the response of Mumbai soil available at Tarapura Nuclear Power plant (BARC), JNPT and Mumbai port, ground response analyses were conducted by Desai and Choudhury (2015b). Synthetic ground motions matching with uniform hazard spectra for different level of ground motions were generated and used as bedrock motion by Desai and Choudhury (2015b). Based on the work, Desai and Choudhury (2015b) found that while maximum amplification at JNPT was found in the frequency range of 1.75–2.25 Hz, BARC site experienced maximum amplification in the higher frequency range of 10.75–11.38 Hz. During another work, in order to understand the effect of ground motion parameters on soil response, equivalent linear site response analyses of 11 typical sites in Kolkata was attempted by Chatterjee and Choudhury (2016). A total of four input motions were selected as bedrock

motion having wide range of acceleration, frequency content and duration. Based on the analyses, it was concluded that 2001 Bhuj EQ even though having low bedrock motion amplitude but caused more amplification due to high frequency content as well as duration in comparison to 1995 Kobe EQ having high bed rock motion amplitude.

2 Frequency Content of Ground Motion

Any periodic function P(t), with period T_0 can be represented as the summation of harmonic functions using complex Fourier series. Though acceleration time history generated during an EQ is not a periodic function, it can be assumed to be one having infinite period. In such a case, an EQ excitation can be represented as (Chopra 2014);

$$P(t) = \sum_{j=-\infty}^{\infty} P_j e^{i(j\omega_0 t)} \qquad (1)$$

$$P_j = \frac{1}{T_0} \int_0^{T_0} P(t) e^{-i(j\omega_0 t)} dt \quad j = 0, \pm 1, \pm 2\ldots \qquad (2)$$

where $P(t)$ is the excitation in the time domain and P_j is the Fourier amplitude of the excitation, $\left(\omega_0 \left(\omega_0 = \frac{2\pi}{T_0}\right)\right)$ is the lowest frequency present in the motion. Equations (1) and (2) represent two sided expression for Fourier Transformation where both the negative and positive frequencies are considered. Equations (1) and (2) above represent Fourier transformation pair. The array P_j is the Fourier transformation of the excitation $P(t)$ while function $P(t)$ is the inverse Fourier transformation of the sequence P_j. It can be seen from Eq. (2) that, $P_j = P_{-j}^*$, where, asterisk indicates complex conjugate. It has to be highlighted here that the above equations are applicable when $P(t)$ is a well-defined function. In discrete Fourier Transform (DFT) however, where the signal in time domain is sampled at equal time intervals, the harmonic representation of the signal can be given as (Chopra 2014);

$$P_j = \frac{1}{T_0} \sum_{n=0}^{N-1} P_n e^{-i(j\omega_0 t_n)} \Delta t \qquad (3)$$

$$P_n = \sum_{j=0}^{N-1} P_j e^{i(j\omega_0 t_n)} \qquad (4)$$

where P_j is the DFT of the time domain signal and P_n is the inverse DFT of the array P_j, N is the number of elements present in the original signal P_n, Δt is the sampling interval of the original signal and $t_n = n\Delta t$. The time domain signal P_n can be presented in frequency domain by a Fourier spectrum which can be obtained by plotting the array P_j against frequency $(j\omega_0)$. Figure 1 shows the Fourier amplitude spectrum of motion 11 where the motion is decomposed into its constitutive harmonics of different frequencies. It has to be highlighted here that unlike Eqs. (1) and (2), only positive frequencies are considered in Eqs. (3) and (4) such that these equations become one sided Fourier transformation. Likewise the original Fourier transformation, harmonics corresponding to the $N/2 < j < N - 1$ are complex conjugate of the harmonics corresponding to $0 < j \le N/2$. Hence, the maximum frequency present in harmonic representation of the signal is $N/2\ \omega_0$. This frequency is known as Nyquist frequency (ω_{nyq} or $f_{nyq} = \frac{\omega_{nyq}}{2\pi}$) or Folding frequency (Chopra 2014). The value of f_{nyq} for a ground motion record is determined as half of the sampling rate (inverse of the sampling interval). In majority of available site response studies, f_{nyq} of the input motion has been given very little consideration. Rather, input motions beyond certain range of frequency are curtailed while performing such site response analysis. In deconvolution analysis however, considering the input motion up to f_{nyq} in SHAKE analysis show amplification in ground motion at
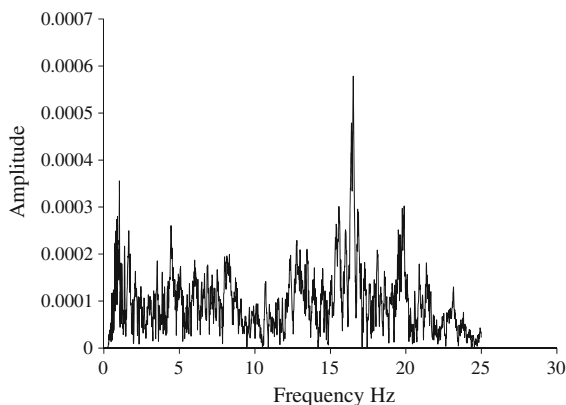
bedrock in comparison to the reference site. Such reduction in the amplification factor lesser than unity at higher frequencies was also reported by Masuda et al. (2001) while analyzing vertical seismic array records during 1987 Chibakentoho-oki earthquake. Such an observation is a clear indication that the amplitude of ground motion enhances with depth, which is impractical. As per Yoshida (2015), such divergence in the deconvolution was also reported in Japan when frequencies up to f_{nyq} were considered in the analysis. In the present work however, unlike previous studies related to deconvolution, the effect of higher frequencies up to f_{nyq} in traditional site response analysis carried out using SHAKE2000 is assessed.

3 Input Soil Properties

For the present analysis, one typical borelog is taken out of 41 boreholes from Kumar et al. (2016) as presented in Table 1. Presented borelog belongs to a client site from Delhi which was selected for the construction of a public utility as mentioned by Kumar et al. (2015, 2016). Specific information about the location of the site was not given in the original papers (Kumar et al. 2015, 2016) as it was part of a private project. It can be observed from Table 1 that the surface layer consists of fill in a thickness of 4.1 m. Below the fill layer, alternate layers of silty sand and low compressibility clay are encountered till 30 m depth. In order to examine soil properties available, disturbed and undisturbed soil samples were collected from various depths as per Kumar et al. (2016). Depth of water table for the borehole was reported as 2.1 m after observing the possible fluctuation for 24 h (Kumar et al. 2015, 2016). Plasticity index of the soil was found to be varying from 10 to 24% as highlighted by Kumar et al. (2016). Similarly, Iyenger and Ghosh (2004) based on studies by the Geological Survey of India also highlighted that the depth of bedrock varies over across the Delhi. Rock outcrop is visible at places like Link road, Daryaganj, Pusa road. In contrast, depth of bedrock at Patel road and Rajghat is of the range 40–60 m, which dips to a depth of 100 m at Houz Khas and further dips to 150 m near the River Yamuna. Collectively, a difference of 150 m in bedrock depth can be observed throughout the city. Soil type obtained in the present study are found


Fig. 1 Fourier amplitude spectrum of input motion 11

Table 1 Typical borelog considered in the present analysis

BH	Ground water table at 2.1 m below GL				
Depth below Gl (m)	Soil description	Thickness of layer	Soil classification	Depth (m)	SPT-N
1	Fill	4.15			
2					
3					
4					
5	Clay with low compressibility	1.5	CL	4.15	9
6				5.65	10
7	Silty sand	3.5	SM		
8				7.15	17
9					
10				9.15	24
11	Clay with low compressibility	6	CL		
12				11.15	17
13				12.65	19
14					
15				14.15	21
16				15.15	28
17	Silty sand	6	SM	16.65	34
18					
19				18.15	38
20				19.65	42
21					
22				21.15	50
23	Clay with low compressibility	3	CL	22.65	44
24					
25				24.15	48
26	Silty sand		SM	25.65	55
27					
28				27.15	62
29				28.65	75
30				30.15	88

similar to previous studies which also confirmed the existence of silty sand and low compressibility clay in Delhi region. Iyenger and Ghosh (2004) highlighted that northern part of Delhi lithology consists of clay with trace of silt and sand. Similarly, borelog report by MoES (Ministry of Earth Science 2014) suggested existence of alternate layers of clay and sand up to a depth of 30 m for entire Delhi city. SPT is a widely used in situ test in a borehole to evaluate the geotechnical properties of soil. During the test, a split

spoon sampler with external and internal diameter of 50 and 35 mm respectively and 650 mm long is driven into the soil under the impact of a 63.5 kg hammer from a height of 760 mm. The sampler is driven to penetrate to a depth of 450 mm. The number of blows required by the sampler to penetrate the last 300 mm of depth is called the SPT-N value. As per IS 2131 (1981), the SPT should be done at 1.5 m interval and whenever SPT-N exceeds 50 for 300 mm penetration, it should be reported as refusal. Further, as per IS

Fig. 2 Suitability of inbuilt G_{max}-N correlation in SHAKE2000 for the present analysis (Kumar et al. 2016)
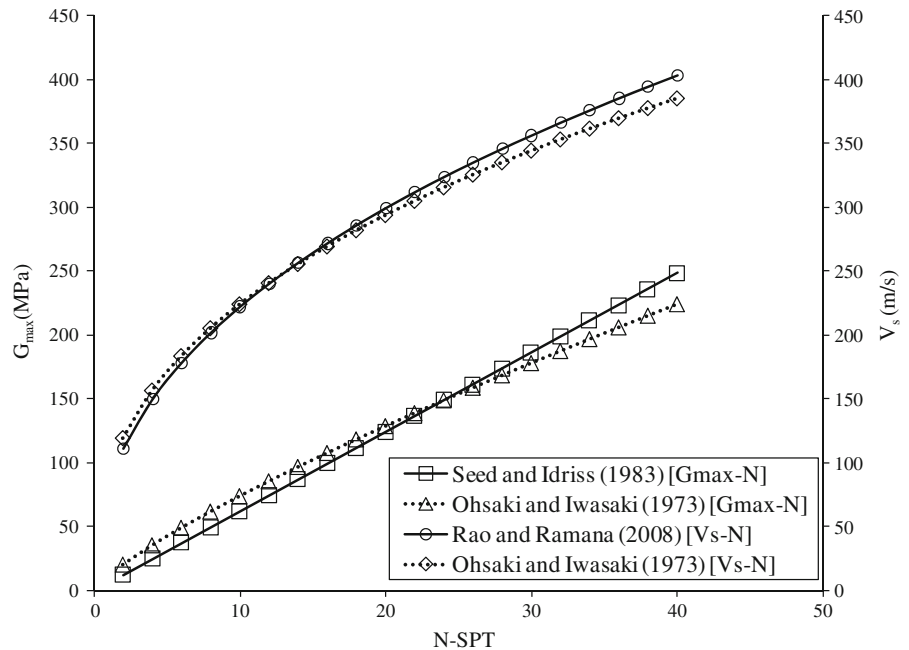
1982: (1974), the test should be done in 150 mm diameter borehole. In addition to collection of soil samples from different depths, SPT tests were also performed in all the 41 boreholes. One typical borelog is shown in Table 1 with SPT-N values with depth. Based on N-SPT variation with depth, average SPT-N value for this borelog is found out to be 18.8. National Earthquake Hazard Reduction Program (NEHRP, BSSC 2003) developed site classification system based on average SPT-N value of the top 30 m layer (N_{30}). According to NEHRP, this borehole belongs to site class D ($15 < N_{30} < 50$). It has to be highlighted here that study area of Delhi lies in seismic zone IV (IS 1893: 2002).Collectively, presence of soft soil layers (N-SPT < 15)having low plasticity along with shallow water table as obtained from the borelog (Table 1) highlights that the site may undergo liquefaction during future earthquakes. Thus a separate study is needed to explore the liquefaction potential of the soil and is beyond the scope of this work.
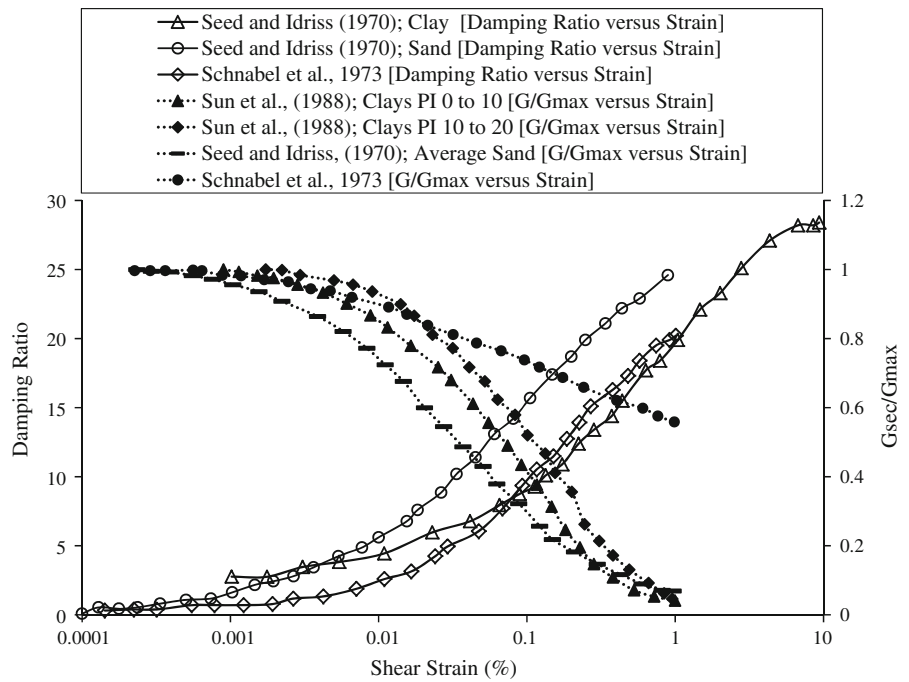
4 Dynamic Soil Properties

In equivalent linear site response analysis, strain compatible shear modulus (G) and damping ratio (β) values are used in order to approximate nonlinear soil

behavior. To initiate the analysis procedure, low strain shear modulus (G_{max}) and β values are required to be given as input. For the present purpose, β value of 5% is taken throughout the soil column. In order to calculate G_{max} a number of in-built equations are available in SHAKE2000. These equations correlate G_{max} with different other strength parameters such as; N-SPT, effective overburden pressure (σ'_v), coefficient of earth pressure at rest (K_0), mean pressure (σ_m), shear wave velocity (V_s), unit weight (γ_t), acceleration of gravity (g), undrained shear strength (S_u), void ratio (e) etc. For the present work however, value of G_{max} should be estimated from N-SPT obtained from borelog. At present no correlation between G_{max} and SPT-N value is available for Delhi site (Kumar et al. 2016). Following in-built correlation given by Seed et al. (1983) in SHAKE2000 is used in the present work.

$$G_{max} = 65N \tag{5}$$

To justify the use of Eq. (5) for Delhi region, Kumar et al. (2016) performed two step verification. In first step, the correlation between V_s and SPT-N proposed by Rao and Ramana (2008) for Delhi site was compared with the correlation developed by Ohsaki and Iwasaki (1973). It was found that both the correlations were matching for the entire range of

Fig. 3 Dynamic properties of soils used in the present analysis (Kumar et al. 2016)

Legend:
- △ Seed and Idriss (1970); Clay [Damping Ratio versus Strain]
- ○ Seed and Idriss (1970); Sand [Damping Ratio versus Strain]
- ◇ Schnabel et al., 1973 [Damping Ratio versus Strain]
- ▲ Sun et al., (1988); Clays PI 0 to 10 [G/Gmax versus Strain]
- ◆ Sun et al., (1988); Clays PI 10 to 20 [G/Gmax versus Strain]
- ━ Seed and Idriss, (1970); Average Sand [G/Gmax versus Strain]
- ● Schnabel et al., 1973 [G/Gmax versus Strain]

SPT-N values (Fig. 2). This suggested that, both the correlations were developed for similar type of soil conditions. In second step, correlation proposed by Ohsaki and Iwasaki (1973) between SPT-N value and G_{max} was compared with Eq. (5) (Fig. 2). Figure 2 shows that, both the correlations are matching well again for the entire SPT-N range. Therefore, Kumar et al. (2016) concluded that in-built correlation in SHAKE2000 can be used confidently for Delhi region.

Another important input parameter in site response analysis is dynamic soil properties. Soil is a complex material which exhibits nonlinear behavior even at small strain level also. As the strain increases, G value reduces while βvalue increases suggesting both modulus reduction (G/G_{max}) and β are function of shear strain (γ). G/G_{max} and β curves are different for different soil types and are obtained from detailed laboratory experiments for each type of soil. Commonly used methodologies to determine above dynamic material curves are simple shear, torsional shear, cyclic triaxial and resonant column test (Dorourdian and Vucetic 1995; Stewart et al. 2001). However, due to lack of such facilities available on regional level, use of standard material curves for each type of soil is widely practiced (Stewart et al. 2001). Such standard curves can be chosen for site response analysis based on soil type, confining pressure,

plasticity index (PI), over consolidation ratio (OCR) and many other parameters. In general, three types of soils are encountered as observed from borelogs. These soils include silty sand, low compressibility clays and medium compressibility clays. For silty sand layers average G/G_{max} and β curves for average sand developed by Seed and Idriss (1970) are used. Seed and Idriss (1970) developed G/G_{max} and β curves based on detailed field and laboratory testing on sand from California region. Similarly, Sun et al. (1988) studied G/G_{max} ratio of clay with different PI with over consolidation ratio (OCR) of 5–15. Sun et al. (1988) found that low value of PI has considerable effect on the position of G/G_{max} curve when compared to high PI clays. Sun et al. (1988) proposed different G/G_{max} curve for clay with different plasticity Index (PI) values. Hence, G/G_{max} curve for clay soil is selected from Sun et al. (1988) based on PI values. The average damping curve for clay as per Seed and Idriss (1970) is used for both the clays (CI and CL) since damping curve is independent of PI of the clays. Further, based on SPT values, soil below 30 m depth is found very dense and thus G/G_{max} and β curves for very dense soil proposed by Schnabel (1973) are used for the present analysis referring to earlier published work (Chen 1997; Kim and Sitar 2007; Rao and Ramana 2009). Selected G/G_{max} and β curves for the present analyses

Table 2 Characteristics of selected input ground motions (predominant frequency is considered to be the frequency corresponding to the peak Fourier amplitude)

S. no.	Ground motion details as per SHAKE2000	Epicentral distance (km)	Magnitude	PGA (g)	Duration (s)	Predominant frequency (Hz)	Sampling rate (Hz)
1	ADAK, ALASKA 1971-M 6.8;R-67KM, N81E	86.77	6.8	0.098	24.58	3.32	200
2	ANCHORAGE, ALASKA 1875, M-6, R81-GOULE HALL STATION	81.93	6	0.036	18.59	5.42	200
3	ANCHORAGE ALASKA 1975, M 6, R 79, WESTWARD HOTEL STATION (BASEMENT)	78.37	6	0.049	38.96	1	200
4	ANZA 02/25/80, BORREGO AIR BRANCH 225	43.1	5.3	0.046	10.25	2.39	200
5	ANZA 02/25/80 1047, TERWILLIGER VALLEY 135	15.8	5.3	0.08	10.01	6.54	200
6	BISHOP-ROUND VALLEY 11/23/84 1914, MCGEE CREEK SURFACE 270	42.35	5.8	0.075	6.8	3.9	200
7	BORREGO MOUNTAIN 04/09/68 0230, EL CENTRO ARRAY 9, 270	60	6.4	0.056	39.95	0.46	100
8	BORREGO MOUNTAIN 04/09/68 0230, PASADENA-ATHENAEUM, 270	216.8	6.4	0.009	60.23	0.61	200
9	BORREGO MOUNTAIN 04/09/68 0230, TERMINAL ISLAND, 339	205	6.4	0.008	51.8	2.5	200
10	CAPE MENDOCINO EARTHQUAKE RECORD 04/25/92, M_w-7.0, 90 DEG COMPONENT	10	7.1	1.03	59.98	4.44	50
11	CHALFANT 07/20/86 1429, BISHOP PARADISE LODGE,070	19.8	6.4	0.046	39.95	16.5	200
12	CHILE EARTHQUAKE, VALPARAISO RECORD, 3/3/85	129.2	7.8	0.12	79.39	2.1	200
13	COALINGA 05/02/83 2342 PARKFIELD, FAULT ZONE 6/ 090	43.9	6.5	0.055	39.95	0.43	100
14	COALINGA 05/09/83 PALMER AVE ANTICLINE RIDGE, 090	12.5	5.3	0.215	40	2.29	200
15	GEORGIA, USSR 06/15/91 0059, BAZ X	49	6.2	0.033	34.07	1.22	200
16	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER 230	15.9	5	0.1	19.88	1.41	200
17	KERN COUNTY 7/21/52 11:53, SANTA BARBARA COURTHOUSE 042	80.5	7.5	0.086	75.35	1.84	100
18	KOBE 01/16/95 2046, ABENO 000	24.9	6.9	0.22	139.98	0.26	100
19	KOBE 01/16/95 2046, KAKOGAWA 000	22.5	6.9	0.25	40.91	0.91	100
20	KOBE 01/16/95, KOBE PORT ISLAND 090	0.9	6.9	0.53	42	0.79	100
21	LIVERMORE 01/27/80 0233, HAYWARD CSUH STADIUM 236	33.9	5.8	0.027	15.98	3.61	200
22	LIVERMORE 01/27/80 0233 LIVERMORE MORGAN TERR PARK 265	20.6	5.8	0.197	24	5.61	200
23	LOMA PRIETA TA 10/18/89 00:05, ANDERSON DAN DOWNSTREAM 270	16.9	7	0.24	39.59	2.14	200
24	LOMA PRIETA TA 10/18/89 00:05, HOLLISTER DIFF ARRAY 255	13.9	7	0.27	40	1.48	200

Table 2 continued

S. no.	Ground motion details as per SHAKE2000	Epicentral distance (km)	Magnitude	PGA (g)	Duration (s)	Predominant frequency (Hz)	Sampling rate (Hz)
25	MICHIOACAN EARTHQUAKE 19/9/85, CALETA DE CAMPOS, N-COMPONENT	38.36	8.1	0.14	81.06	1.39	200
26	NORTHERN CALIFORNIA 09/22/52 1141, FERNDALE 134	44.3	5.2	0.07	40	1.31	200
27	NORTHRIDGE EQ 1/17/94 1231, ANACAPA ISLAND	71.4	6.7	0.013	40	4.46	50
28	NORTHRIDGE EQ 1/17/94 1231, ARLETA 360	9.5	6.7	0.31	39.94	1.46	50
29	PARKFIELD 06/28/66 04:26, CHROME # 8	11.2	6.1	0.116	26.09	0.85	100
30	TRINIDAD 11/08/08, 10:27, RIO DEL OVERPASS E	72	7.2	0.13	22	3.14	200

are shown in Fig. 3. Depending upon the soil encountered in the borehole, suitable curves as discussed above is assigned. Table 1 presents a typical borelog having only two types of soils with no medium compressibility clay present. For this reason, only two soil types assigned while modeling this borelog. Similarly, other boreholes are modeled.

5 Ground Motion Selection

In any site response analysis, bedrock motion is very important factor. Being a developing nation with low to very high seismic activity, ground motion recording in India started only after 1980. Since then, no great EQ has taken place in the country. In the absence of regional earthquake data, selection of standard ground motions such as 1940 El-Centro, 1985 Mexico, 1989 Loma Prieta, 1994 Northridge, 1995 Kobe and 1999 Chi-Chi etc. have been practiced worldwide and in India (Phanikanth et al. 2011; Kumar et al. 2016). In addition, available regional EQ motions are scaled up to meet the response spectrum at the site of interest. Further, generation of synthetic ground motions based on uniform hazard spectra and hazard value for site response analysis is also acknowledged worldwide (Kennedy et al. 1984; Deodatis 1996; Bazzurro et al. 1998; Papageorgiou et al. 2000; Stewart et al. 2002; Krawinkler et al. 2003; Mavroeidis et al. 2004; Kramer and Mitchell 2006; Anbazhagan et al. 2010). Selecting one or two input motions based on seismic hazard analysis is not enough to represent ground

motion properties in terms of peak ground acceleration (PGA), duration and frequency content. Similarly, selecting specific EQ generated motions which are recorded somewhere else may have limited application. In order to account for the uncertainties related to ground motion properties, a large set of bedrock motions should be considered (Kumar et al. 2016). Selection of records should be made such a way that, they consist of a wide variation in all three ground motion properties namely PHA, frequency content and duration. Apart from this, magnitude of earthquake and epicentral distance also should be given due attention while selecting input motions. Motions should be selected such a way that, they consist of both near and far field sources (Kumar et al. 2016). Kumar et al. (2015, 2016) clearly highlighted that amplitude of bedrock motion controls the amplification in ground motion between the bedrock and the surface. Thus, in the absence of any regional ground motion record, it might not be feasible to randomly select bedrock motion. Hence, to have a better understanding about the response of the site specific soil, a large set of ground motions covering a wide range of ground motion characteristics should be considered. Ground motion characteristics which control the earthquake response are amplitude, duration and frequency content. Highlighting that study area of Delhi is susceptible to seismic hazard both due to local as well as regional active sources, Kumar et al. (2015, 2016) shortlisted 30 globally recorded ground motion. In the present analysis, all the 30 globally recorded earthquake motions are chosen from PEER

Table 3 PGA variation with depth considering input motions up to default value of 15 Hz

Layer number

Motion number	1	2	3	4	5	6	8	10	12	13	15	16	17	18	19
1	0.30	0.19	0.17	0.18	0.18	0.16	0.13	0.10	0.10	0.09	0.08	0.09	0.09	0.09	0.09
2	0.11	0.06	0.05	0.05	0.05	0.05	0.05	0.06	0.05	0.05	0.03	0.03	0.03	0.03	0.03
3	0.20	0.14	0.11	0.10	0.09	0.10	0.11	0.10	0.09	0.09	0.07	0.06	0.05	0.05	0.05
4	0.17	0.14	0.11	0.11	0.10	0.09	0.08	0.07	0.06	0.06	0.06	0.06	0.05	0.05	0.05
5	0.28	0.15	0.10	0.11	0.12	0.13	0.15	0.15	0.13	0.12	0.10	0.10	0.09	0.09	0.09
6	0.22	0.11	0.12	0.14	0.15	0.13	0.14	0.14	0.12	0.10	0.07	0.07	0.06	0.07	0.08
7	0.17	0.15	0.14	0.13	0.13	0.11	0.09	0.08	0.06	0.06	0.05	0.05	0.05	0.06	0.06
8	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
9	0.05	0.05	0.04	0.04	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01
10	1.01	0.60	0.57	0.61	0.55	0.56	0.68	0.72	0.74	0.79	0.76	0.84	0.91	0.91	0.84
11	0.11	0.07	0.06	0.06	0.06	0.06	0.06	0.06	0.05	0.05	0.04	0.03	0.03	0.03	0.03
12	0.30	0.20	0.18	0.18	0.17	0.16	0.15	0.14	0.13	0.13	0.12	0.12	0.11	0.11	0.10
13	0.14	0.12	0.12	0.11	0.11	0.10	0.09	0.09	0.08	0.07	0.07	0.06	0.06	0.06	0.06
14	0.52	0.42	0.38	0.39	0.41	0.38	0.29	0.25	0.22	0.21	0.19	0.19	0.20	0.21	0.21
15	0.07	0.07	0.06	0.06	0.05	0.05	0.04	0.03	0.03	0.03	0.02	0.02	0.02	0.02	0.02
16	0.19	0.16	0.14	0.13	0.11	0.11	0.11	0.11	0.12	0.11	0.10	0.10	0.09	0.09	0.09
17	0.25	0.23	0.21	0.20	0.18	0.16	0.14	0.13	0.11	0.11	0.10	0.09	0.09	0.09	0.09
18	0.51	0.38	0.34	0.33	0.30	0.26	0.23	0.21	0.20	0.19	0.21	0.21	0.22	0.22	0.22
19	0.62	0.50	0.42	0.35	0.36	0.34	0.28	0.25	0.24	0.23	0.22	0.22	0.24	0.25	0.25
20	0.57	0.51	0.48	0.48	0.48	0.48	0.49	0.51	0.52	0.52	0.51	0.52	0.52	0.53	0.52
21	0.16	0.12	0.10	0.09	0.09	0.08	0.07	0.06	0.05	0.05	0.03	0.03	0.03	0.03	0.03
22	0.64	0.41	0.29	0.26	0.27	0.32	0.35	0.34	0.31	0.28	0.24	0.22	0.21	0.20	0.19
23	0.61	0.51	0.47	0.43	0.38	0.33	0.32	0.30	0.28	0.27	0.26	0.25	0.24	0.24	0.23
24	0.78	0.72	0.67	0.63	0.58	0.53	0.44	0.39	0.34	0.33	0.31	0.30	0.29	0.28	0.28
25	0.43	0.38	0.34	0.30	0.27	0.25	0.23	0.20	0.18	0.17	0.16	0.16	0.15	0.14	0.14
26	0.28	0.22	0.20	0.20	0.19	0.18	0.15	0.12	0.10	0.09	0.09	0.08	0.08	0.08	0.07
27	0.08	0.06	0.06	0.05	0.05	0.04	0.03	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01
28	0.58	0.50	0.46	0.42	0.38	0.37	0.39	0.41	0.41	0.39	0.37	0.35	0.33	0.31	0.29
29	0.22	0.12	0.16	0.19	0.17	0.11	0.15	0.14	0.11	0.11	0.10	0.11	0.11	0.10	0.09
30	0.40	0.32	0.30	0.29	0.26	0.22	0.17	0.15	0.13	0.12	0.12	0.12	0.13	0.13	0.13

database as given in SHAKE2000. Details of selected ground motions are given in Table 2. It can be observed from Table 2 that widely used ground motions such as 1995 Kobe EQ, 1989 Loma Prieta EQ and 1985 Mexico EQ etc. are the part of selected ground motions in the present work for site response analysis. Further, it can be observed from Table 2 that, variation in ground motion characteristics is quite a wide. A wider range of amplitude (0.036–1.03 g) is covered considering possible range of bedrock PHA for the site which can be obtained by the seismic

hazard study. Similarly, the duration of the selected ground motions are varying from as low as 6.8 s to as high as 140 s as shown in Table 2. Epicentral distances of the selected motions also vary from 0.9 km to as much as 216 km representing a region seismically active due to nearby as well as distant sources. Similarly, magnitude of selected ground motions vary in the range of 5.0–8.1 (M_w). Predominant frequency of each of the selected ground motions are determined as the frequency corresponding to peak of Fourier amplitude spectra are shown in the

Table 4 PGA values considering cut off frequency as f_{nyq}

Motion number	1	2	3	4	5	6	8	10	12	13	15	16	17	18	19
Layer number															
1	0.30	0.19	0.18	0.19	0.18	0.17	0.14	0.11	0.10	0.10	0.09	0.10	0.10	0.11	0.12
2	0.10	0.07	0.07	0.06	0.06	0.06	0.06	0.06	0.06	0.05	0.04	0.03	0.04	0.04	0.04
3	0.20	0.14	0.10	0.10	0.09	0.10	0.11	0.10	0.09	0.09	0.07	0.06	0.05	0.05	0.05
4	0.17	0.14	0.11	0.11	0.10	0.08	0.08	0.07	0.06	0.06	0.06	0.05	0.05	0.05	0.05
5	0.27	0.18	0.14	0.10	0.11	0.15	0.17	0.15	0.16	0.16	0.13	0.11	0.09	0.08	0.08
6	0.24	0.15	0.15	0.15	0.16	0.14	0.15	0.15	0.13	0.11	0.11	0.10	0.09	0.09	0.09
7	0.17	0.15	0.14	0.13	0.13	0.11	0.09	0.08	0.06	0.06	0.05	0.05	0.05	0.06	0.06
8	0.02	0.02	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
9	0.05	0.05	0.04	0.04	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01
10	0.99	0.54	0.55	0.56	0.55	0.53	0.66	0.78	0.88	0.85	0.81	0.90	0.82	1.03	1.03
11	0.16	0.10	0.10	0.08	0.08	0.09	0.10	0.08	0.06	0.07	0.07	0.06	0.06	0.05	0.05
12	0.30	0.22	0.18	0.17	0.17	0.16	0.16	0.15	0.15	0.14	0.14	0.14	0.14	0.14	0.12
13	0.14	0.12	0.12	0.11	0.11	0.10	0.09	0.09	0.08	0.07	0.07	0.06	0.06	0.06	0.06
14	0.52	0.43	0.38	0.39	0.41	0.38	0.29	0.25	0.22	0.21	0.19	0.19	0.20	0.20	0.22
15	0.07	0.07	0.06	0.06	0.05	0.05	0.04	0.03	0.03	0.03	0.02	0.02	0.02	0.02	0.02
16	0.19	0.16	0.14	0.13	0.11	0.10	0.11	0.12	0.11	0.11	0.10	0.10	0.10	0.10	0.10
17	0.25	0.23	0.21	0.20	0.19	0.16	0.14	0.13	0.11	0.11	0.10	0.09	0.09	0.09	0.09
18	0.51	0.38	0.34	0.32	0.30	0.27	0.23	0.21	0.20	0.20	0.22	0.22	0.23	0.22	0.22
19	0.63	0.50	0.41	0.36	0.35	0.35	0.29	0.26	0.24	0.23	0.22	0.23	0.24	0.25	0.25
20	0.57	0.51	0.49	0.49	0.48	0.48	0.50	0.51	0.52	0.53	0.52	0.52	0.53	0.53	0.53
21	0.16	0.12	0.10	0.09	0.09	0.08	0.07	0.06	0.05	0.04	0.03	0.03	0.03	0.03	0.03
22	0.66	0.41	0.28	0.26	0.28	0.33	0.36	0.34	0.30	0.28	0.23	0.22	0.21	0.20	0.20
23	0.61	0.51	0.47	0.43	0.38	0.33	0.32	0.29	0.28	0.27	0.26	0.25	0.25	0.25	0.24
24	0.78	0.72	0.67	0.63	0.58	0.53	0.45	0.39	0.34	0.33	0.31	0.30	0.29	0.28	0.28
25	0.42	0.38	0.34	0.30	0.27	0.26	0.23	0.20	0.18	0.18	0.16	0.16	0.15	0.15	0.14
26	0.28	0.22	0.20	0.20	0.19	0.18	0.15	0.12	0.10	0.10	0.09	0.08	0.08	0.08	0.07
27	0.08	0.06	0.05	0.05	0.05	0.04	0.04	0.03	0.03	0.02	0.02	0.02	0.01	0.01	0.01
28	0.58	0.50	0.46	0.42	0.39	0.37	0.39	0.40	0.40	0.39	0.37	0.35	0.33	0.32	0.31
29	0.24	0.14	0.15	0.18	0.18	0.13	0.17	0.16	0.13	0.11	0.11	0.11	0.11	0.11	0.12
30	0.41	0.32	0.31	0.29	0.26	0.22	0.17	0.15	0.13	0.12	0.12	0.13	0.13	0.13	0.13

penultimate column of Table 2. For each ground motion, sampling rate is also listed in the last column, Table 2. Based on sampling interval, value of f_{nyq} for each of the selected 30 ground motions are determined.

6 Analyses

In the present work, two sets of site response analyses for the soil column (typical borelog is shown in Table 1) are carried out. In each set, soil profile and in-situ densities are modeled in SHAKE2000 following the details given in Table 1. All soil columns are modeled to be resting on elastic halfspace for the present analyses. Layers having thickness greater than 3 m are subdivided into sublayers having smaller thicknesses. In each set, all the modeled soil columns are subjected to above selected 30 ground motions and their responses are observed. Based on the soil characteristics, appropriate dynamic soil properties as discussed earlier are assigned to each soil layer. Initial values of G_{max} and β are chosen as discussed in the preceding sections. Further, in each set, each of the

Table 5 PGA comparison (ratio of PGA obtained in Table 4 to PGA obtained in Table 5)

Motion number	Layer number														
	1	2	3	4	5	6	8	10	12	13	15	16	17	18	19
1	1.00	1.01	0.97	0.98	0.97	0.96	0.94	0.97	0.95	0.95	0.90	0.85	0.84	0.83	0.76
2	1.09	0.86	0.75	0.86	0.88	0.79	0.91	0.96	0.91	0.92	0.92	0.79	0.63	0.71	0.73
3	1.01	0.99	1.02	1.01	1.01	0.99	1.01	1.01	1.00	1.00	0.99	1.00	0.96	1.00	0.98
4	1.00	0.98	1.03	1.00	0.99	1.01	1.00	1.00	1.02	1.00	1.02	1.02	1.02	1.00	1.00
5	1.04	0.81	0.74	1.06	1.07	0.83	0.89	0.98	0.80	0.76	0.78	0.85	1.00	1.11	1.13
6	0.93	0.75	0.83	0.91	0.93	0.94	0.95	0.88	0.89	0.88	0.68	0.69	0.71	0.85	0.90
7	1.00	1.00	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	1.00	0.98	1.00
8	1.10	1.00	1.00	0.95	0.97	0.99	0.96	0.99	0.98	1.00	1.01	1.01	1.01	1.00	1.00
9	0.94	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00	0.93	1.00	1.01	1.00	1.00
10	1.02	1.10	1.05	1.10	1.01	1.05	1.03	0.93	0.84	0.93	0.93	0.94	1.11	0.88	0.81
11	0.67	0.70	0.64	0.76	0.83	0.65	0.58	0.76	0.84	0.70	0.54	0.49	0.55	0.69	0.74
12	1.00	0.91	0.98	1.01	1.00	0.98	0.94	0.95	0.89	0.90	0.86	0.83	0.80	0.76	0.81
13	1.00	1.01	0.99	1.00	1.00	1.00	1.00	1.00	1.01	1.00	1.00	1.00	1.00	1.02	1.01
14	1.00	0.98	1.00	1.01	1.00	1.00	1.01	1.00	1.01	1.01	0.97	1.01	0.99	1.04	1.00
15	1.00	1.00	1.00	1.00	0.98	1.00	0.97	1.00	1.04	0.96	1.00	0.95	0.95	0.94	1.00
16	1.01	0.99	0.98	0.99	0.99	1.04	0.96	0.99	1.02	1.02	1.02	0.95	0.88	0.88	0.89
17	1.00	1.00	0.98	1.00	0.97	0.99	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00
18	1.00	1.00	1.00	1.03	1.00	0.96	1.00	1.00	0.99	0.96	0.95	0.97	0.95	0.96	0.97
19	0.98	1.00	1.01	0.98	1.01	0.99	0.98	0.98	1.00	1.01	1.02	0.99	0.99	0.99	0.99
20	1.00	1.00	0.98	0.98	1.00	0.99	0.98	0.99	0.98	0.99	0.98	0.99	0.97	0.99	0.98
21	1.01	1.00	1.00	0.99	0.99	1.00	1.00	1.00	1.02	1.02	1.00	1.00	1.00	1.00	1.00
22	0.98	1.00	1.01	0.98	0.97	0.96	0.97	1.00	1.03	1.03	1.04	1.02	0.99	0.97	0.96
23	1.00	1.00	1.00	1.00	1.00	1.01	1.01	1.01	0.99	0.99	1.00	0.99	0.97	0.97	0.94
24	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.99	0.99	1.01	1.01	1.00	1.01	1.01	1.01
25	1.01	0.99	1.00	1.00	0.99	0.98	1.00	1.02	0.98	0.98	0.99	0.99	0.97	0.98	0.99
26	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	0.99	0.99	0.99	1.00	1.00	1.00
27	1.00	1.02	1.02	1.02	0.98	0.97	0.96	1.00	0.98	0.99	0.96	0.92	1.05	1.03	1.00
28	1.00	0.99	1.00	1.00	0.98	1.00	0.99	1.02	1.02	1.00	1.01	1.02	1.00	0.97	0.95
29	0.90	0.82	1.07	1.05	0.98	0.85	0.86	0.87	0.90	0.94	0.92	0.99	0.96	0.92	0.75
30	0.99	0.99	0.99	1.00	1.00	1.00	0.99	1.01	1.02	1.02	0.98	0.97	0.97	0.98	0.98

30 input motions is assigned at the top of the bottommost layer and responses in terms of PGA values at the top of each sublayer are obtained for all the boreholes. In first set of analysis, all the 30 input motions are selected up to maximum frequency of 15 Hz (by default set in SHAKE2000) and the response of all the boreholes are observed. In the second analysis however, all the 30 input motions are considered up to f_{nyq} of the respective motion and the responses of all the boreholes are observed.

6.1 Results and Discussions

Based on first set of analyses results with ground motion considered up to default frequency of 15 Hz, PGA values obtained at each sublayer for all the input motions for one typical borehole are tabulated in Table 3. It can be observed from Table 3 that, surface PGA values vary from 0.02 to 1.01 g while the bedrock motion show peak horizontal acceleration (PHA) variation from 0.01 to 0.84 g. Similarly, for the
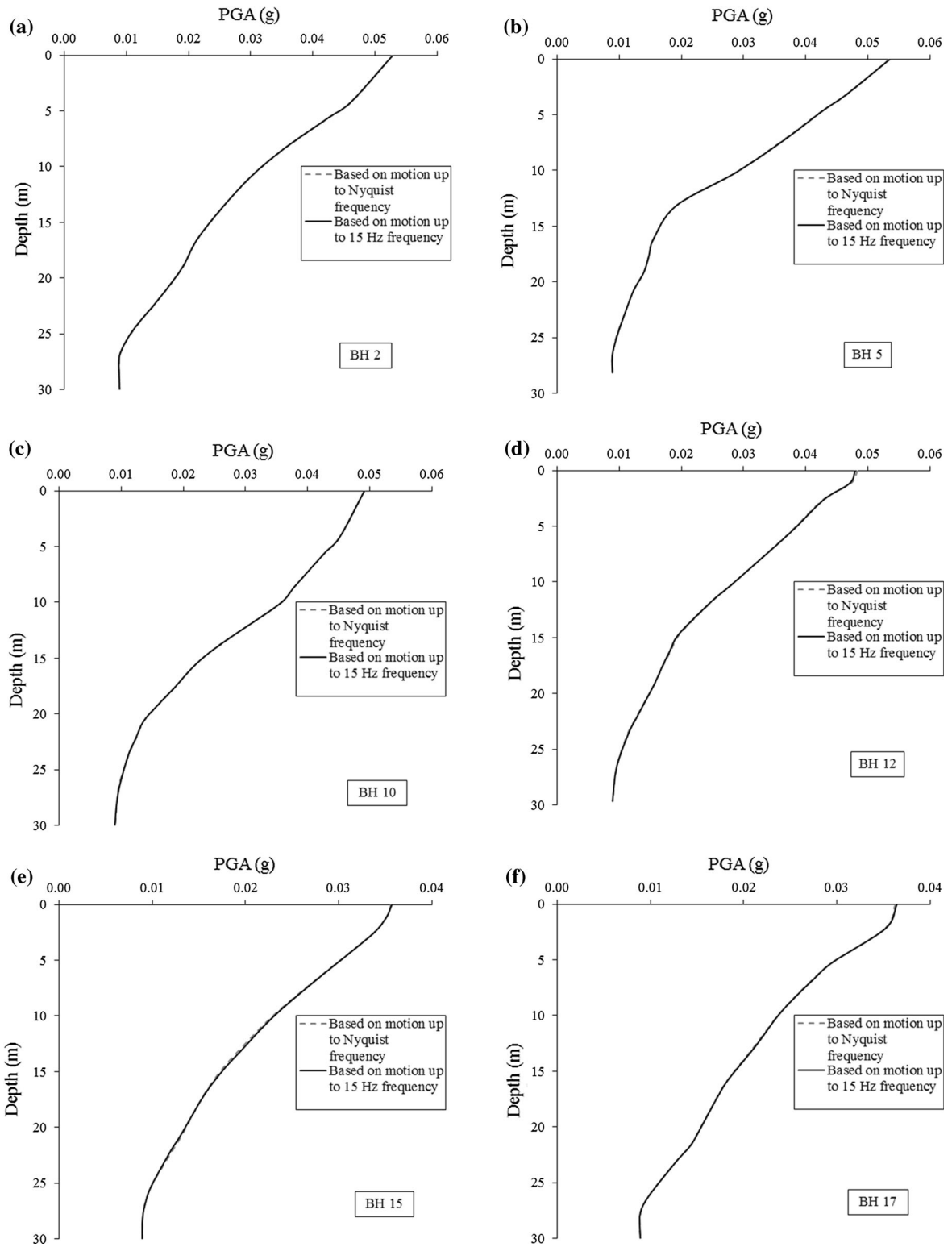
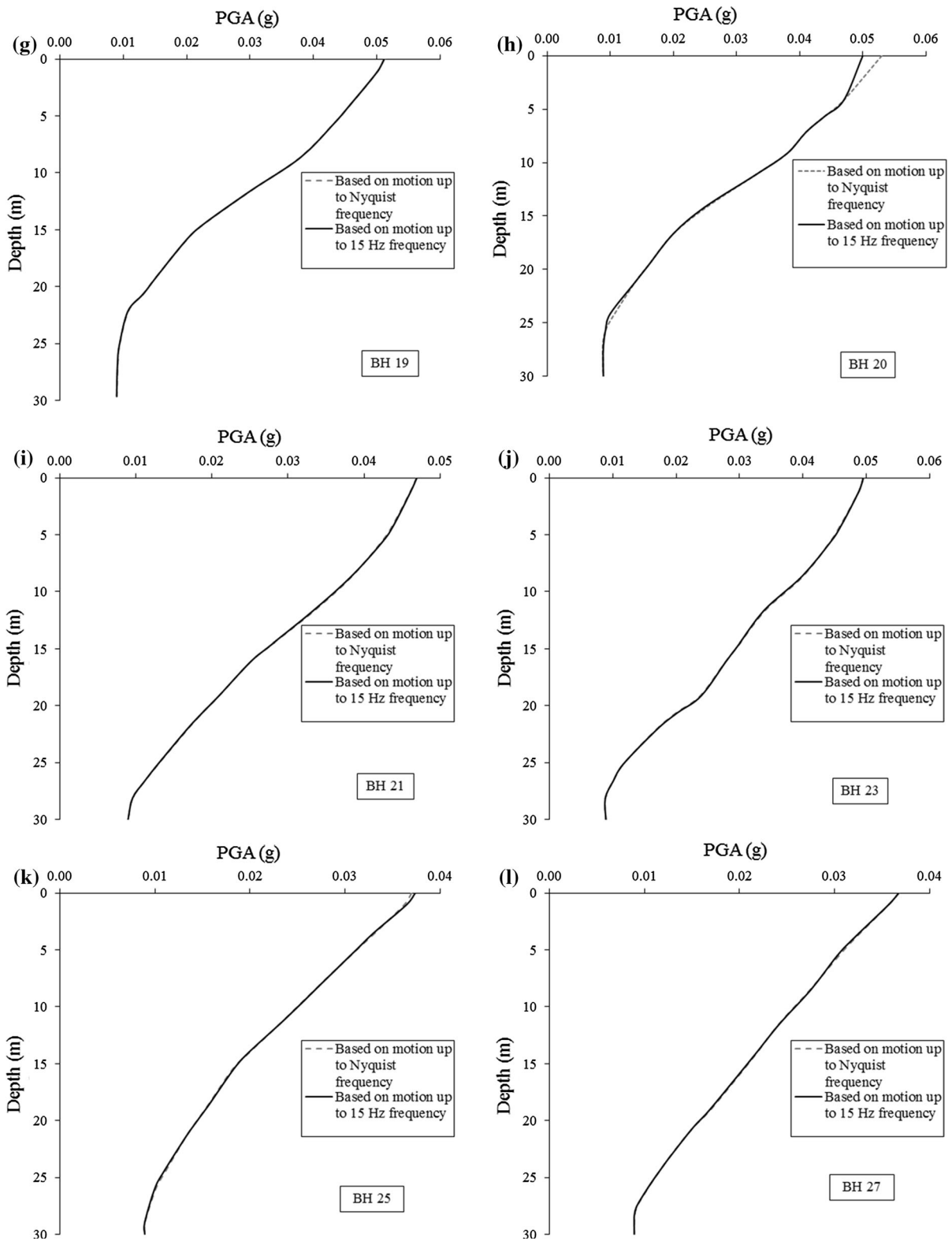**Fig. 4** **a–p** PGA comparison with depth for input motion 9
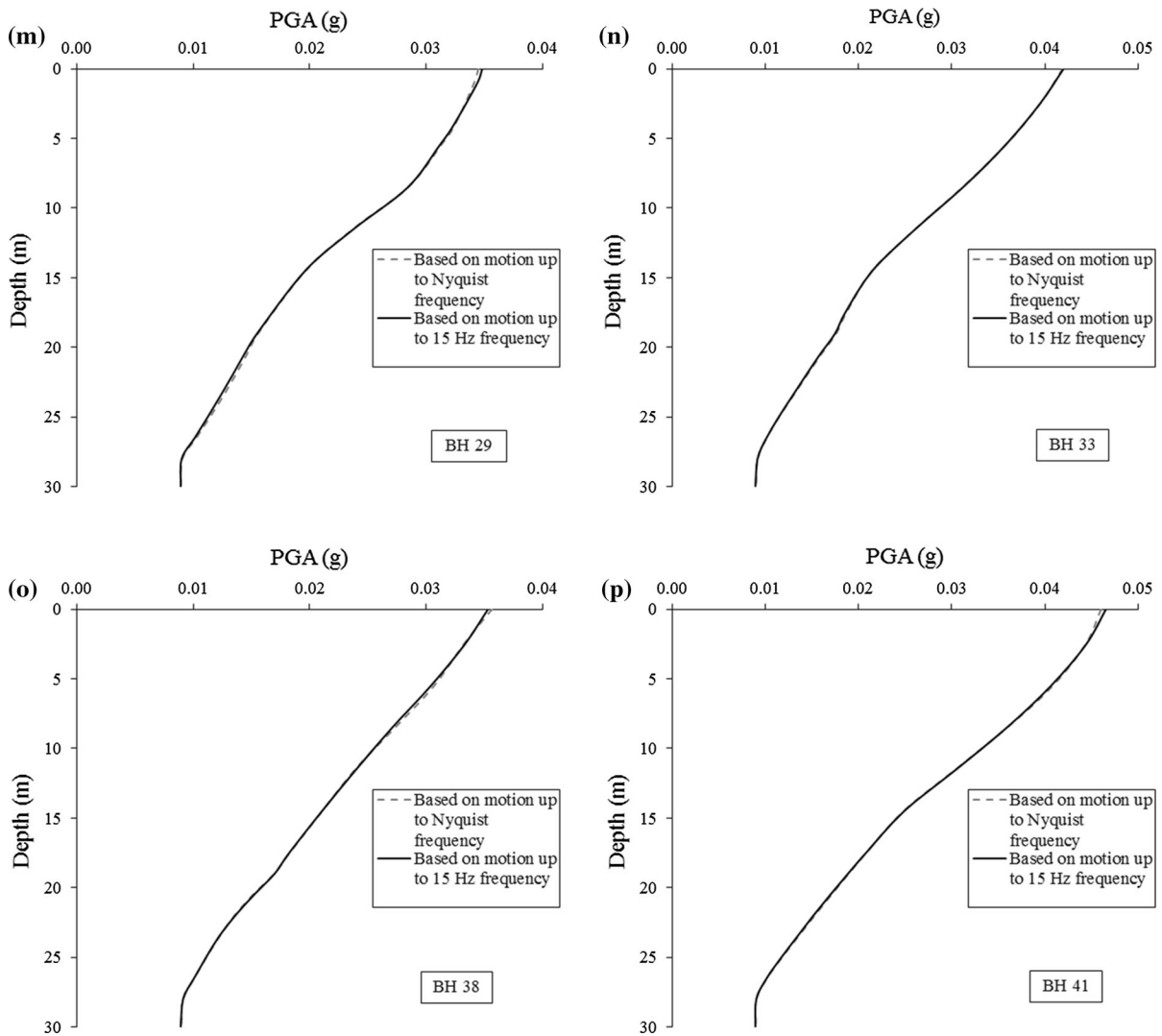
Fig. 4 continued

Fig. 4 continued

second set of analyses with input motions considered up to f_{nyq}, PGA variation with depth (ref. Table 3) is presented in Table 4. It can be observed from Table 4 that obtained surface PGA values are in the range 0.02–0.99 g not varying considerably in comparison to surface PGA listed in Table 3. However, bedrock PHA values obtained by considering default frequency of 15 Hz or all the motions (Layer 19, Table 3) are found smaller than the bedrock PHA values obtained by considering frequency up to f_{nyq} for all the input motions (Layer 19, Table 4). In order to explore this change in PGA with depth between two sets of analyses results, ratio of PGA at each sublayer

obtained in Table 3 to PGA at the corresponding layer obtained in Table 4 are listed in Table 5. It can be observed from Table 5 that changing the maximum frequency in SHAKE2000 from the default value of 15 Hz to f_{nyq} has negligible effect on the obtained PGA values except for certain input motions (motion 11 and 6, Table 5). For majority of input motions, the ratios of two sets of PGA at most of the sublayers are close 1.0. However at certain sublayers, the ratio of PGA values also shows variation in the range of 0.63–1.11. In order to understand the reason for values close to 1 and other values at specific layers, input motion 9 and 10 are considered here. Figure 4a–p
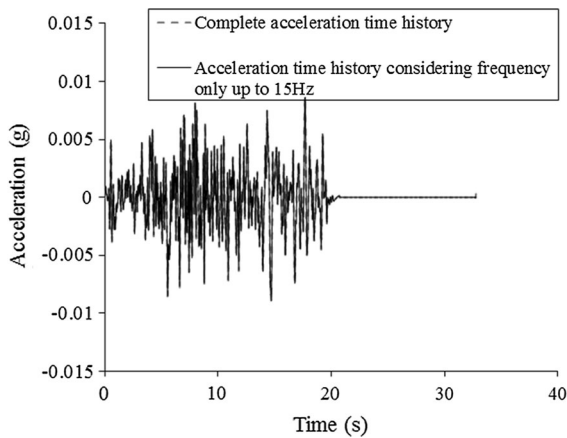
Fig. 5 Acceleration time history of motion 9
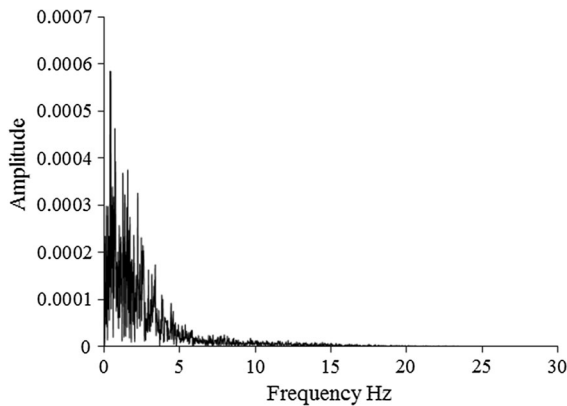
Fig. 6 Fourier amplitude spectra for input motion 9
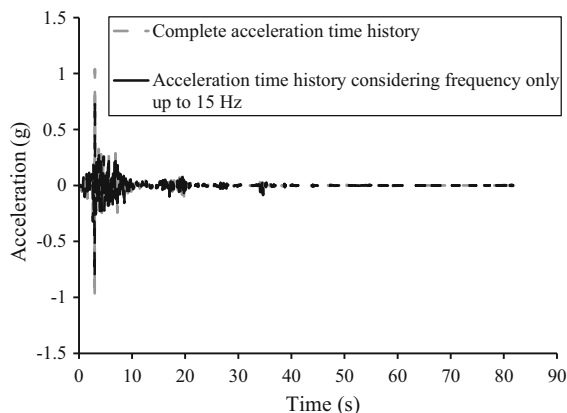
Fig. 7 Acceleration time histories for input motion 10

present the PGA variation with depth for motion 9 considering two sets of analyses for 16 boreholes. It can be observed from Fig. 4a–p that PGA variations

with depth from two sets of analyses using input motion 9 are identical throughout the depth for all the boreholes. In order to understand this, plot of complete actual acceleration time history and acceleration time history considering motion up to 15 Hz for input motion 9 are shown in Fig. 5. It can be observed from Fig. 5 that the ground motion signatures obtained from two dataset are identical. In order to understand the effect of frequency content of actual input motion, corresponding Fourier spectrum for input motion 9 is presented in Fig. 6. It can be observed from Fig. 6 that frequency components of actual motion beyond 10 Hz are negligible compared to the frequency components within 10 Hz frequency range. Therefore, considering maximum frequency as default value of 15 Hz or f_{nyq} does not cause any significant change in response of same soil column.

At certain sub-layers as shown in Table 5, the ratio of two PGA values are also found lesser than 1. In order to understand this, complete acceleration time history and acceleration time history considering motion up to 15 Hz for input motion 10 is shown in Fig. 7. It can be observed from Fig. 7 that due to frequency consideration only up to 15 Hz, input motion gets significantly modified in comparison to complete acceleration time history. This change in the ground motion might be the attribute for change in PGA other than unity at certain sublayers. Similar changes are observed for all the boreholes. However, considering length of the paper, sub-layer wise PGA and PGA ratio variation with depth are only presented here for one typical borehole.

Further, based on Table 5 it can be observed that for input motions 6 and 11, the ratio of two sets of PGA are never converging to a value of unity for any sublayer. For input motion 11, PGA variation with depth based on two sets of analyses for 16 boreholes presented earlier (corresponding to Fig. 4a–p are shown in Fig. 8a–p. It is understood from Fig. 8a–p that the two sets of PGA values are considerably different throughout the depth for all the 16 boreholes. Hence, it can be concluded from Fig. 8a–p that the choice of maximum frequency as default value of 15 Hz may not be the wise option every time. To understand further, actual acceleration time history for input motion 11, is plotted in Fig. 9 along with acceleration time history considering motion up to 15 Hz. It can be observed from Fig. 9 that similar to Fig. 7, while frequency consideration only up to
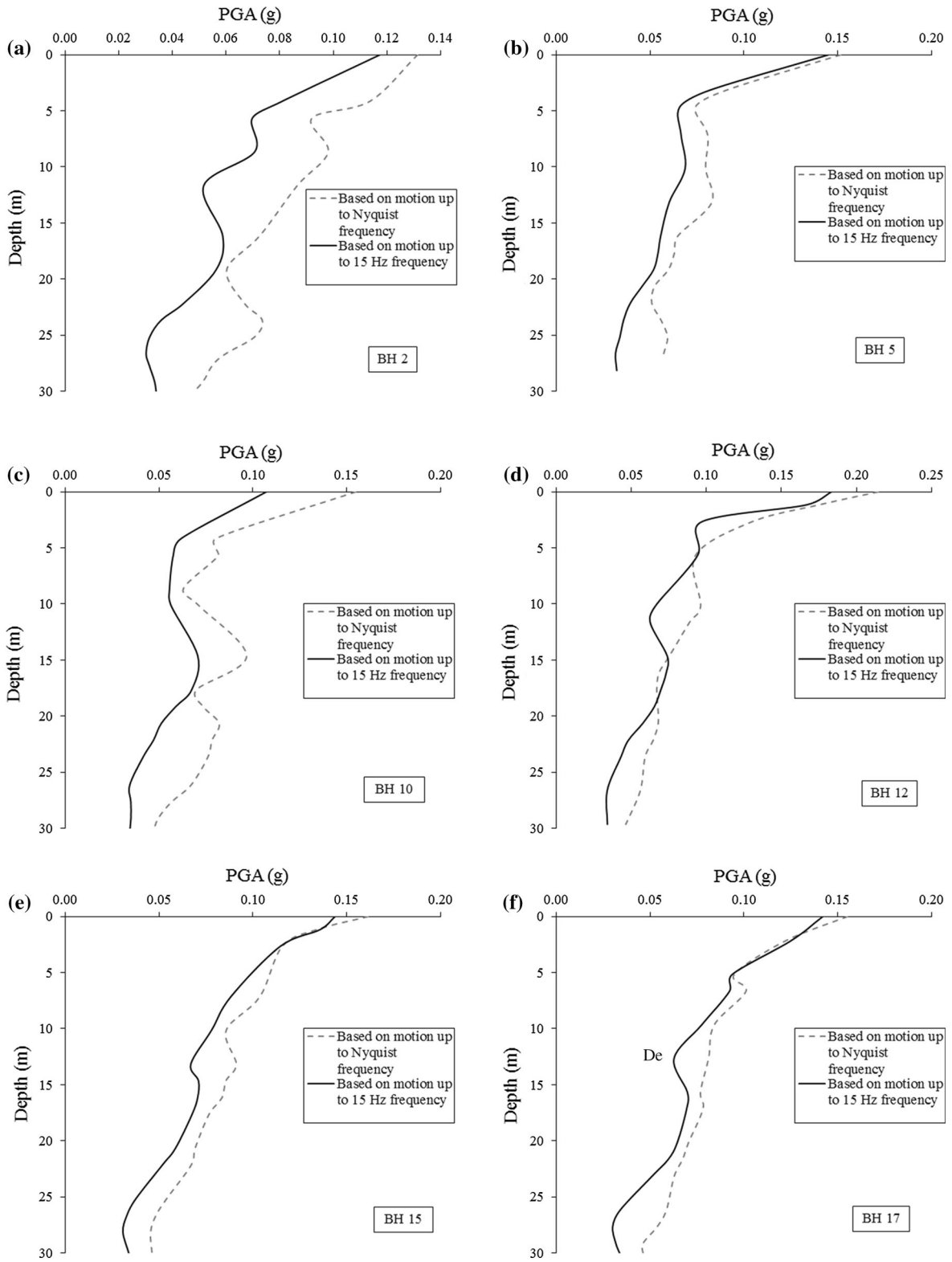
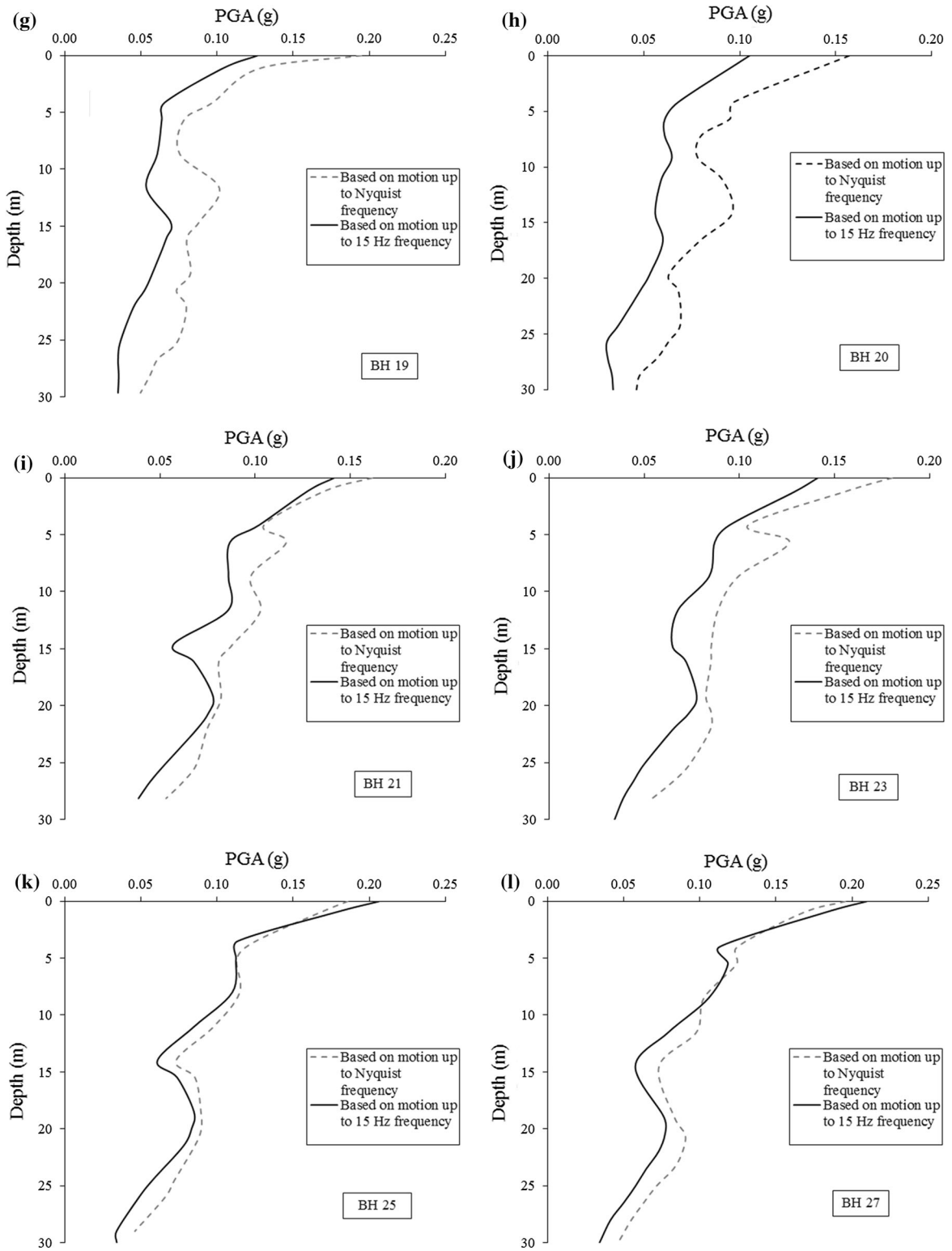Fig. 8 a–p PGA comparison with depth for input motion 11

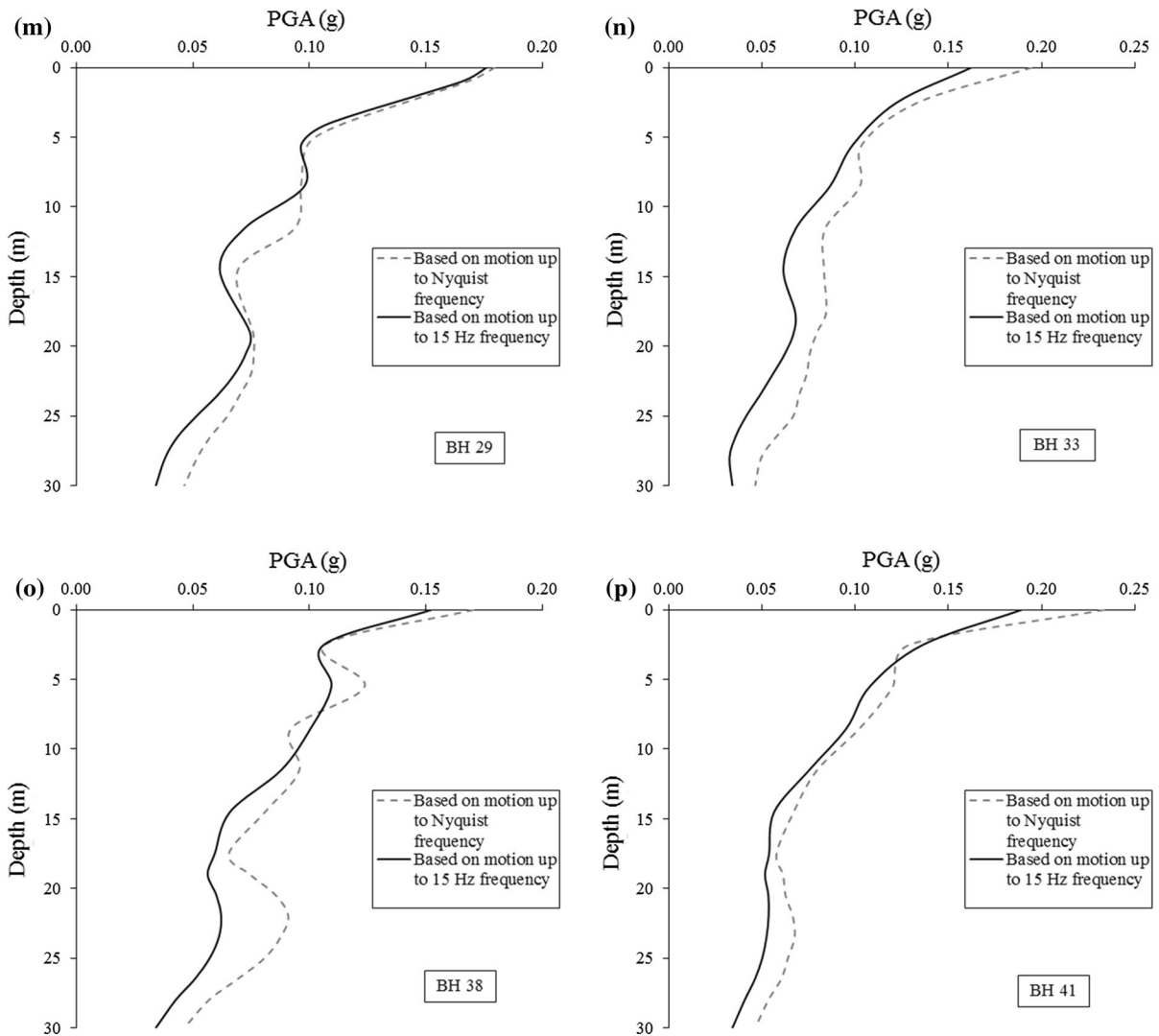**Fig. 8** continued

(m)

(n)

(o)

(p)

Fig. 8 continued

15 Hz, input motion gets significantly modified, which is the reason behind deviation in the ratio of PGA at all the sublayers. Further, Fourier spectrum for input motion 11 already shown in Fig. 1 suggests that the predominant frequency for input motion 11 is 16.5 Hz. Thus, in case acceleration time history is considered only up to 15 Hz for this motion, core frequency component is ignored throughout the analysis. For this reason, the ratios of two PGA values are lesser than unity for all the sub-layers.

Similar difference in two sets of PGA with depth can be obtained for input motion 6. In order to explore the reason, the Fourier spectrum of input motion 6 is shown in Fig. 10. It can be observed from Fig. 10 that a significant part of frequency component exists even up to 25 Hz frequency. Hence, in case frequency components beyond 15 Hz are neglected, corresponding PGA values are lesser in comparison to the PGA values obtained considering motion up to f_{nyq}. Similar observations can also be made for input motion 2 (Table 5).

Collectively, based on the above analyses, it can be said that PGA values are underestimated when maximum frequency in SHAKE2000 is considered up to 15 Hz. Miura et al. (2000) highlighted this shortcoming of SHAKE stating the reason that SHAKE analysis
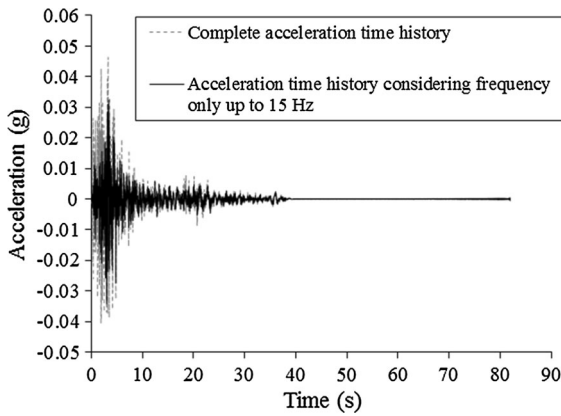
Fig. 9 Actual acceleration time histories for motion 11
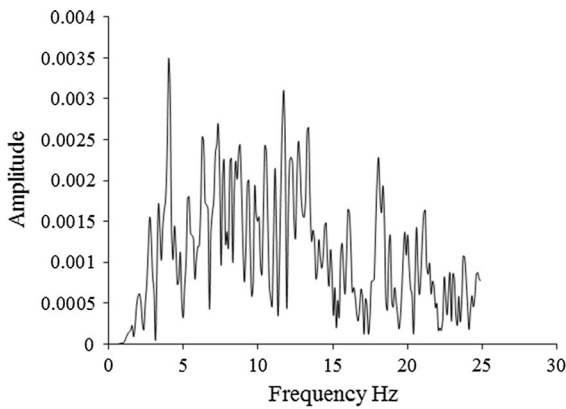
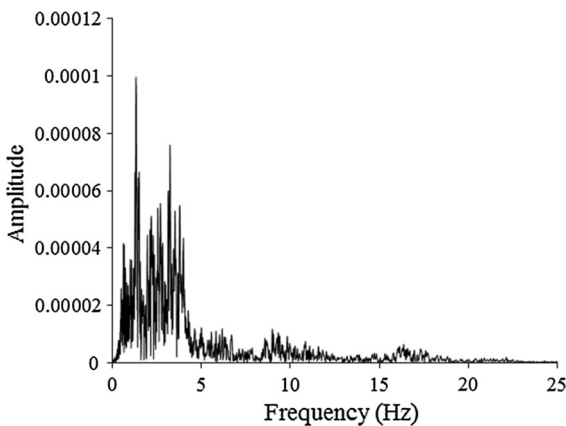Fig. 10 Fourier amplitude spectrum of input motion 6

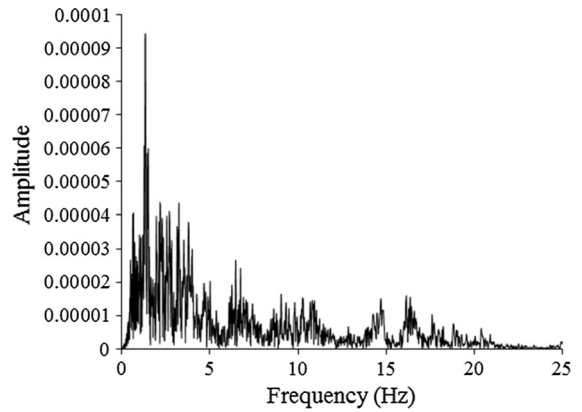Fig. 11 Fourier spectrum for Alum rock site at 5.7 m depth

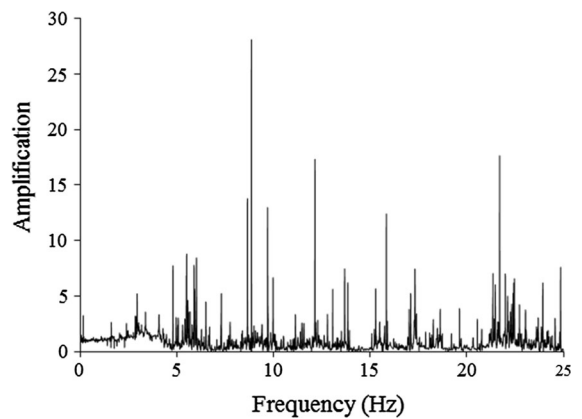Fig. 12 Fourier spectrum for Alum rock site at the surface

Fig. 13 Amplification spectrum for Alum rock site

overestimation of β particularly at higher frequency content collectively can cause underestimation of soil response. Further, Miura et al. (2000) validated this observation by comparing results from site response analysis and the actual soil response observed from vertical seismic array in Tokyo Bay area. In addition, Miura et al. (2000) clearly mentioned that such an underestimation of soil response at higher frequency content based on SHAKE analysis is not specific to any location but can be observed for any site.

6.2 A Typical Case Study Validating Ground Motion Amplification Even at Higher Frequency (Greater Than 10–15 Hz)

Above analyses clearly indicates that in case ground motion is considered up to default value of 15 Hz, it may lead to underestimation of high frequency component for Delhi region. To validate this

is governed by G/G_{max} and β values corresponding to effective strain which is directly related to peak strain. As a result of which, underestimation of G/G_{max} and

observation, geotechnical array recording from center for engineering strong motion data (CESMD) is considered in the present work. Strong motion array recording is taken from Alameda - Posey and Webster geotechnical array. CESMD is a cooperative center for earthquake data established by US Geological Survey (USGS) and California Geological survey (CGS). The center includes strong motion data from USGS National strong motion project, CGS California strong motion instrumentation program and the Advanced Nation Seismic System (ANSS). Both raw and processed data can be obtained from CESMD. Alameda-Posey and Webster geotechnical array is one among many geotechnical arrays which are monitored by CESMD. This geotechnical array is located at 37.7896 N and 122.2766 W. The subsoil consists of deep alluvium. The 30 m average shear wave velocity (V_{s30}) as 208 m is reported in the CESMD website (http://www.strongmotioncenter. org/) for this site suggesting NEHRP site class D. Four seismographs are installed at surface, 5.7, 13.3 and 40 m depth at this station. Figure 11 presents Fourier spectrum of AlumRock_30Oct2007 N-S component at 5.7 m depth below the surface recorded at Alameda-Posey and Webster geotechnical array. Similarly, Fig. 12 presents Fourier spectrum for the same motion at the ground surface obtained from ground motion record. Based on Fourier spectrum at ground surface and at 5.7 m depth presented in Figs. 11 and 12 respectively, amplification factor is obtained as shown in Fig. 13. It can be observed from Fig. 13 that considerably higher amplification values are possible even after 15 Hz. Similar comparison was also shown by Miura et al. (2000) based on vertical seismic array at Tokyo Bay area as discussed earlier. Considerable amplification at higher frequency content obtained from geotechnical array records are consistent with the observations made earlier based on SHAKE2000 analysis in this paper.

7 Conclusion

SHAKE2000 is world widely followed software for equivalent site response analysis. In absence of regional ground motion record, due importance is given to the amplitude of ground motion consistent with regional hazard values. However, while using selected ground motions for site response analysis in

SHAKE2000, frequency content are not given proper attention. Further, a default value of maximum frequency as 15 Hz is generally considered without assessing higher frequency contents. In the present work, 41 boreholes from Delhi regions are analyzed for 30 globally recorded ground motions having wide range of ground motion parameters. Each borehole is subjected to all the 30 ground motions and for each motion two sets of ground response analyses are performed. While one set of analyses use ground motion up to default frequency of 15 Hz as given in SHAKE2000, other set of analyses used ground motions up to f_{nyq}. Based on the present work, it is concluded that same ground motions but with different values of maximum frequency content used in SHAKE2000 analysis can significantly change the response of same soil column depending upon the frequency content of input motion. In case, input motion are significant in higher frequencies (<15 Hz) range, ignoring such part of input motion in the ground response analysis will lead to underestimation of soil response at various depths. These findings are consistent with limited extisting literature. Further, these observations are also validated by means of recorded ground motions using geotechnical seismic array. The maximum frequency content of motion can be determined from sampling rate. This information should be carefully used in ground response analysis since depending upon the frequency content, analyses can lead to underestimation of same soil column.

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