

Equivalent-Linear Seismic Ground Response Analysis of Some Typical Sites in Mumbai

V. S. Phanikanth · Deepankar Choudhury ·
G. Rami Reddy

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Abstract Any earthquake event is associated with a rupture mechanism at the source, propagation of seismic waves through underlying rock and finally these waves travel through the soil layers to the particular site of interest. The bedrock motion is significantly modified at the ground surface due to the presence of local soil layers above the bedrock beneath the site of interest. The estimation of the amplifications in ground response due to the local soil sites is a complex problem to the designers and the problem is more important for mega cities like Mumbai in India, where huge population may get affected due to devastations of earthquake. In the present study, the effect of local soil sites in modifying ground response is studied by performing one dimensional equivalent-linear ground response analysis for some of the typical Mumbai soil sites. Field borelog data of some typical sites in Mumbai city viz. Mangalwadi site, Walkeswar site, BJ Marg near Pandhari Chawl site are considered in this study. The ground responses are observed for

range of input motions and the results are presented in terms of surface acceleration time history, ratio of shear stress to vertical effective stress versus time, acceleration response spectrum, Fourier amplitude ratio versus frequency etc. The typical amplifications of ground accelerations considering four strong ground motions with wide variation of low to high MHA, frequency contents and durations are obtained. Results show that MHA, bracketed duration, frequency content have significant effects on the amplification of seismic accelerations for typical 2001 Bhuj motion. The peak ground acceleration amplification factors are found to be about 2.50 for Mangalwadi site, 2.60 for Walkeswar site and 3.45 for BJ Marg site using 2001 Bhuj input motion. The response spectrum along various soil layers are obtained which will be useful for designers for earthquake resistant design of geotechnical structures in Mumbai for similar sites in the absence of site specific data.

Keywords Ground response analysis · Response spectrum · Fourier amplitude ratio · Maximum horizontal acceleration · Mean time period

V. S. Phanikanth · D. Choudhury (✉)
Department of Civil Engineering, Indian Institute
of Technology Bombay, Mumbai 400 076, India
e-mail: dc@civil.iitb.ac.in

V. S. Phanikanth
Architecture and Civil Engineering Division,
Bhabha Atomic Research Centre, Mumbai, India

G. Rami Reddy
Reactor Safety Division, Bhabha Atomic Research
Centre, Trombay, Mumbai 400 085, India

1 Introduction

Structures to be designed for earthquake forces shall consider the rupture mechanism of the nearest fault to the site of interest, propagation of seismic waves through the underlying rock and the soil medium

present above the bedrock at the site location as shown in Fig. 1. The literature available shows that the soil layers beneath the site play an important role in ground motion amplifications than the amplifications due to seismic waves travelling in the rock from the rupture zone to the site with much larger distances (say tens of kilometers of rock) (Kramer 2005). Over the years seismologists and more recently geotechnical engineers worked towards the development of quantitative methods for predicting the ground response and thus developed one-dimensional, two-dimensional and three dimensional ground response analysis methods. By performing the ground response analysis of a given soil deposit, the geotechnical engineer will be able to, evaluate the natural frequencies of the soil deposits, assess ground motion amplification and evaluate acceleration response. These details can further be used for any earthquake resistant design of structures considering the geotechnical parameters along with earthquake motions.

Predicting the ground motion amplifications of a layered soil deposit in the regions where earthquake hazards exist is a challenging task to the geotechnical engineers and the problem becomes more important for a highly populated city like Mumbai in India (Mhaske and Choudhury 2010, 2011). Though few researchers had already carried out the ground response analysis for several cities in India, but the same for Mumbai city is still scarce. For example, in India, Govindaraju et al. (2004); Choudhury and Shukla (2011), Shukla and Choudhury (2011a, b) had obtained the site response analysis for various soil sites in Gujarat, Raghukanth et al. (2008) had obtained ground motion estimates for Guwahati city, Boominathan et al. (2007) had assessed seismic hazard

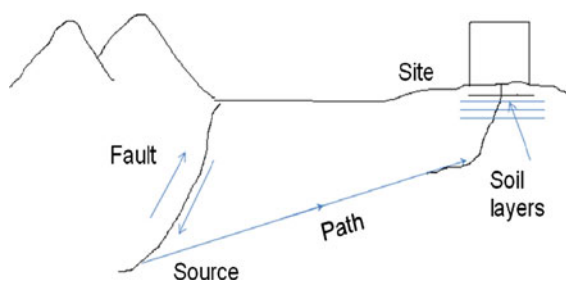


Fig. 1 Complete ground response model to include rupture mechanism at source, propagation of stress waves through rock and soils layers beneath the site of interest

considering local site effects for Chennai city. Mohanty et al. (2007) had attempted seismic microzonation of Delhi and Rajiv Ranjan (2005) did seismic response analysis of Dehradun city in India. In the present study an attempt has been made to evaluate the effect of local soil conditions in modifying the ground response. Equivalent linear ground response analysis using computer based program namely DEEPSOIL is used. Field borehole data of some typical sites of Mumbai are considered for the analysis.

1.1 Parameters Influencing Ground Response Analysis

Ideally, a complete ground response analysis (or site specific ground response analysis) should take into account rupture mechanism at source of an earthquake (source), propagation of stress waves through the crust to the top of bedrock beneath the site of interest (path), influence of local soil conditions above bedrock on the ground surface motion (Kramer 2005).

In reality, several difficulties arise and uncertainties exist as:

- i. Mechanism of fault rupture is very complicated and difficult to predict in advance.
- ii. Crustal velocity and damping characteristics are generally poorly known.
- iii. Nature of energy transmission between the source and site is uncertain.

In practice Seismic hazard analyses (probabilistic or deterministic) is used for predicting bedrock motions at the location of the site. They are dependent primarily on empirical attenuation relationships to predict bedrock motion parameters. Deterministic seismic Hazard analysis is performed considering:

- i) Source characterization, which includes identification, characterization of all earthquake sources which may cause significant ground motion in the site of interest.
- ii) Selection of the shortest distance between the source and site of interest.
- iii) Selection of controlling earthquake i.e., earthquake that is expected to produce the strongest level of shaking.
- iv) Defining the hazard at the site formally in terms of ground motions produced at the site by the controlling earthquake.

Once the underlying bedrock motion is arrived, ground response problem becomes one of determining responses of soil deposit. The ground response is modified by the presence of local soil sites and the important parameters influencing the ground motion are:

- i. Shear wave velocity profile with depth.
- ii. Modulus reduction curve (Variation of shear modulus with strain).
- iii. Damping ratio curve (Variation of damping with strain).

A number of techniques are available for ‘ground response analysis’. The methods differ in the simplifying assumptions made, in the representation of stress–strain relations of soil and in the methods used to integrate the equation of motion. The development of existing methods of dynamic response analysis has been a gradual evolutionary process. This may be attributed to the increasing knowledge about the fundamental behavior of soils under cyclic loading derived from field observations and laboratory testing. The methods can be broadly grouped into the following three categories viz (1) Linear analysis, (2) Equivalent-linear analysis and (3) Nonlinear analysis. For the present study, equivalent-linear analysis has been adopted for its simplicity with sufficient amount of accuracy. Also the method is conservative compared to nonlinear site response analysis based on the works carried out by Rayhani et al. (2008); Kokusho (2004). For equivalent-linear analysis, the nonlinear hysteretic stress–strain properties of sand were idealized by equivalent-linear curve as proposed by Schnabel et al. (1972). The method was originally based on the lumped mass model of sand deposits resting on rigid base to which the seismic motions were applied. Later, this method was generalized to wave propagation model with an energy-transmitting boundary. The seismic excitation could be applied at any level in the new model.

2 Wave Propagation Analysis/Site Amplification

During an earthquake seismic waves are generated and the ground motion parameters such as amplitude of motion, frequency content and duration of the ground motion change as the seismic waves propagate through overlying soil and reach the ground surface.

The phenomenon, wherein the local soils act as a filter and modify the ground motion characteristics, is known as ‘soil amplification’. The manner in which these waves travel is a function of the stiffness and attenuation characteristics of the medium (damping) and will control the effects they produce (amplification/de-amplification). Simplified methods like one-dimensional wave propagation analyses may be used to obtain the soil amplification effects.

2.1 One-Dimensional Wave Propagation Analyses

One-dimensional wave propagation analysis is widely used for ‘ground response analysis’ or ‘soil amplification studies’ as it provides reasonable estimates of ground motion (Choudhury and Savoikar 2009). Also large numbers of commercial computer programs with different soil models are available with the advent of technology such as SHAKE (Schnabel et al. 1972), DEEPSOIL v 3.5 (Hashash et al. 2008) etc.

The following assumptions are considered in this analysis:

- i. The soil layers are horizontal and extend to infinity.
- ii. The ground surface is level.
- iii. The incident earthquake motions are spatially-uniform, horizontally-polarized shear waves, and propagate vertically.

The basic wave equation for uniform damped soil on rigid rock is given as (Kramer 2005)

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial z^2} + \eta \frac{\partial^3 u}{\partial z^2 \partial t} \quad (1)$$

The solution to the above wave equation (1) is of the form (Kramer 2005)

$$u(z, t) = Ae^{i(\omega t + k^* z)} + Be^{i(\omega t - k^* z)} \quad (2)$$

where, $u(z, t)$ is horizontal displacement at a depth z below ground level and at time ‘ t ’ (see Fig. 2); ‘ ρ ’ is the density; ‘ G ’ is the Shear modulus and ‘ η ’ is the viscosity and $= (2G/\omega)\zeta'$. Also ‘ A ’ and ‘ B ’ are amplitudes of waves traveling in the $-z$ (upward) and $+z$ (downward) directions, respectively. ‘ k (Wave number)’ $= \omega/V_s$, where ‘ ω ’ is wave frequency, V_s = shear wave velocity. k^* is the complex wave number and is given by $k^* = k(1.0 - i\zeta)$, where

ζ = Damping ratio. Also the transfer function is given by

$$F_2(\omega) = \frac{1}{\cos\left[\frac{\omega H}{V_s}(1 + i\zeta)\right]} \tag{3}$$

and the amplification function is given as,

$$|F_2(\omega)| = \frac{1}{\sqrt{\cos^2\left(\frac{\omega H}{V_s}\right) + \left[\zeta\left(\frac{\omega H}{V_s}\right)^2\right]}} \tag{4}$$

where H = thickness of the soil layer; The period of vibration (fundamental period) corresponding to the fundamental frequency is given by $T = 4H/V_s$. It can be seen that as the damping ratio increases, due to higher viscosity the amplification reduces.

Similar expressions are also available for layered damped soils on rigid rock as well as for elastic rock (Kramer 2005).

3 Method of Analysis

The solution steps involved in the one dimensional equivalent linear ground response analysis are:

- i) The time history of earthquake ground motion is broken down into sum of series of simple harmonic Fourier loading functions.
- ii) Response is evaluated for each individual Fourier series loadings.

- iii) The responses are combined to get the output time history.

Also the equivalent-linear method is an iterative approach and is defined as below (Kramer 2005).

- a) Initial estimates of G and ζ are made for each layer. The initially estimated values usually correspond to the same strain level. The low strain values are often used for initial estimates.
- b) The estimated G and ζ are used to compute the ground response, including the time histories of shear strain for each layer.
- c) The effective shear strain in each layer is determined from the maximum shear strain in the computed shear strain time history. For layer j .

$\tau_{eff}^{(i)} = R_\gamma \gamma_{max}^{(i)}$ where the superscript refers to the iteration number and R_γ is the ratio of effective shear strain to the maximum shear strain. R_γ depends on the local earthquake magnitude (M) and can be obtained from $R_\gamma = (M-1)/10$.

- d) From this effective shear strain, new equivalent linear values, $G^{(i+1)}$ and $\zeta^{(i+1)}$ are chosen for the next iteration.

Steps ‘b’ to ‘d’ are repeated until the differences between the computed shear modulus and damping ratio in two successive iterations falls below some predetermined values in all layers. Although convergences are not absolutely guaranteed, differences of less than 5–10% are usually achieved in three to five iterations (Schnabel et al. 1972).

The shear modulus and damping ratio curves can be defined either by defining discrete points or by defining the soil properties to be used in the hyperbolic model. The option of defining soil curves using discrete points is only available for equivalent linear analysis. In this option G/G_{max} and damping ratio (ζ) are defined as functions of strain. DEEPSOIL also incorporates the extended hyperbolic model. The modified hyperbolic model developed by Metasovic (1993), is based on the hyperbolic model by Konder and Zelasko (1963) but adds two parameters ‘ β ’ and ‘ s ’ that adjust the shape of the backbone curve. This is represented as:

$$\tau = \frac{G_{mo}\gamma}{1 + \beta\left(\frac{\gamma}{\gamma_r}\right)^s} \tag{5}$$

where G_{mo} is the initial shear modulus; τ is the shear strength and γ is the shear strain. In the present

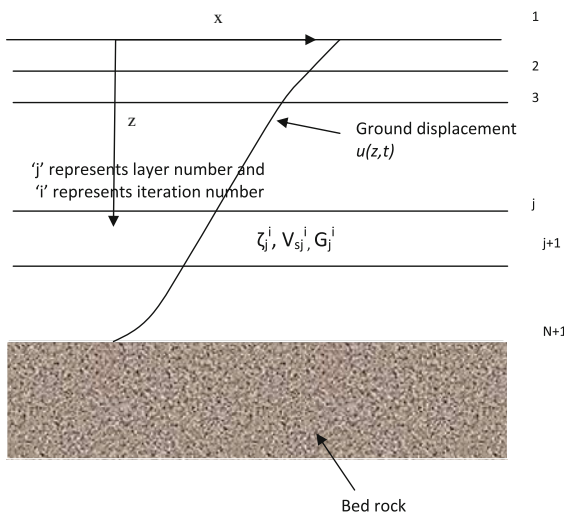


Fig. 2 Ground displacements at any time ‘ t ’ along the depth of layered soil deposits

Table 1 Detail bore log data of Mangalwadi site, Mumbai

Layer no.	Stratum	Layer thickness (m)	Depth below GL (m)	SPT
<i>Bore hole MBH# 1: Mangalwadi site, Mumbai</i>				
1	Filled up soil	1.5	1.5	10
2	Yellowish loose sand	1.5	3.0	12
		1.5	4.5	13
		1.5	6.0	16
		2.0	8.0	20
3	Black clayey soil	2.0	8.0	20
4	Yellowish clayey soil	1.8	9.8	25
5	Greyish hard rock	–	>9.8	–
<i>Bore hole MBH# 2: Mangalwadi site, Mumbai</i>				
1	Filled up soil	1.5	1.5	9
2	Yellowish loose sand	1.5	3.0	12
		1.5	4.5	14
		1.5	6.0	15
		2.0	8.0	16
3	Black clayey soil	2.0	8.0	16
4	Yellowish clayey soil	2.0	10.0	18
5	Greyish hard rock	–	>10.0	–

Table 2 Detail bore log data of Walkeswar site, Mumbai

Layer no.	Stratum	Layer thickness (m)	Depth below GL (m)	SPT
<i>Bore hole WBH# 1: Walkeswar site, Mumbai</i>				
1	Back filled soil	3.5	3.5	11
2	Silty gravel	1.5	5.0	19
		2.0	7.0	22
		3.5	10.5	62
		–	>10.5	–
3	Basalt rock	–	>10.5	–
<i>Bore hole WBH# 2: Walkeswar site, Mumbai</i>				
1	Back filled soil	1.0	1.0	9
		2.5	3.5	6
2	Clay with sandy soil	2.7	6.2	25
3	Fractured rock	–	>6.2	–

problem the reference stress is evaluated based on shear strength parameters based on Mohr–Coulomb. The reference shear strain γ_r is evaluated based on the initial shear modulus and reference shear stress. ‘ β ’ and ‘ s ’ are parameters that adjust the shape of the backbone curve.

A set of material curves have been defined in DEEPSOIL for modulus reduction curves and damping ratio curves for different soils. In case of sands by defining effective vertical stress, the appropriate

modulus reduction curves may be arrived. In case of clays, effective vertical stress and plasticity index is required to be defined, for estimating modulus reduction and damping curves.

Thus for a given time history at the bedrock location and by knowing the dynamic soil characteristics i.e., shear wave velocity profile with depth, modulus reduction and damping curves, the ground motions amplifications may be obtained by performing ground response analysis.

Table 3 Detail bore log data of BJ Marg site near Pandhari Chawl, Mumbai

Layer	Stratum	Layer thickness (m)	Depth below GL (m)	SPT
<i>Bore hole BBH# 1: BJ Marg, Mumbai</i>				
1	Filled up soil	2.0	2.0	3
2	Clayey soil	2.0	4.0	10
		1.5	5.5	14
		1.5	7.0	37
3	Basalt rock	–	>7.0	–
<i>Bore hole BBH# 2: BJ Marg, Mumbai</i>				
1	Filled up soil	2.0	2.0	2
2	Brownish clayey soil	2.0	4.0	8
		1.5	5.5	12
		1.5	7.0	22
3	Basalt rock	–	>7.0	–
<i>Bore hole BBH# 3: BJ Marg, Mumbai</i>				
1	Filled up soil	2.0	2.0	4
2	Brownish clayey soil	2.0	4.0	7
		1.5	5.5	12
		1.5	7.0	22
3	Basalt rock	–	>7.0	–

Table 4 Strong motion parameters for different earthquakes considered for the present analysis

Earthquake strong motion parameters	2001 Bhuj earthquake	1989 Loma Prieta earthquake	1989 Loma Gilroy earthquake	1995 Kobe earthquake
Date of occurrence	26/01/2001	17/10/1989	18/10/1989	17/01/1995
Magnitude, M_w	7.7	6.9	7.0	6.9
Recording station	Passport office building, Ahmedabad	Palo Alto Station 1601	Gilroy Array Sta 1, CA—Gavilan College, Water Tank), CA	KJMA station
Distance from source	230 km	36.3 km	21.8 km	0.6 km
MHA	0.106 g	0.278 g	0.442 g	0.834 g
Predominant period	0.26 s	0.30 s	0.38 s	0.36 s
Mean period	0.603 s	0.652 s	0.391 s	0.641 s
Bracketed duration	69.80 s	29.70 s	16.22 s	21.92 s
Significant duration	15.78 s	11.57 s	3.70 s	8.34 s

4 Ground Response Analysis for Some Typical Mumbai Soil Sites

In the present study one dimensional equivalent linear ground response analysis for some typical Mumbai sites is attempted. Field borehole data from Mangalwadi site near Girgaon (MBH#1, MBH#2), Walkeswar site (WBH#1, WBH#2) and BJ Marg near Pandhari Chawl site (BBH#1, BBH#2 & BBH#3) have been used for the present analysis.

The ground water tables for Mangalwadi and Walkeswar sites have been considered 4.0 m below ground level (GL), whereas for BJ Marg site, the same is considered as 2.5 m below GL. The borelog details for the above sites have been presented in Tables 1, 2 and 3. As shear wave velocity records are not available, the correlation between shear modulus (G) with SPT ' N ' value, as suggested by Ohasaki and Iwasaki (1973) is used for the present analysis and is given by the relation,

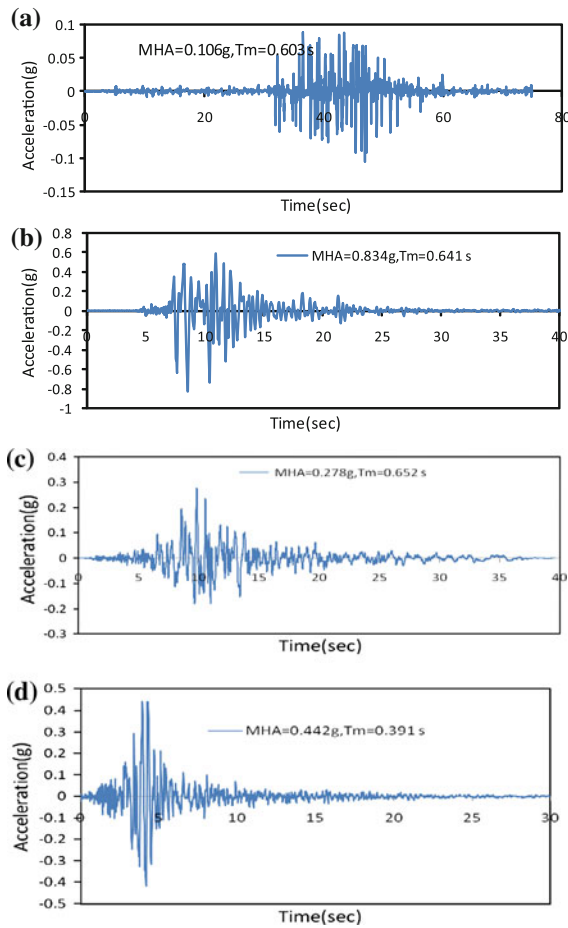


Fig. 3 **a** Acceleration time history of 2001 Bhuj earthquake (Govindaraju et al 2004). **b** Acceleration time history of 1995 Kobe earthquake (Hashash et al. 2008). **c** Acceleration time history of 1989 Loma Prieta earthquake (Hashash et al. 2008). **d** Acceleration time history of 1989 Loma Gilroy earthquake (Hashash et al. 2008)

$$‘G’ \text{ (in kPa)} = 12,000 N^{0.80} \tag{6}$$

The analysis is carried out using geotechnical Software DEEPSOIL v3.5 (Hashash et al. 2008). The bed rock is considered as rigid and hence energy dissipation due to reflection of seismic waves at the bedrock/soil boundaries is not considered.

4.1 Seismic Motions Used for the Present Study

In the present study typical Mumbai soil sites subjected to four strong motion earthquakes with

maximum horizontal accelerations (MHA) of 0.106, 0.278, 0.442 and 0.834 g. The strong motions correspond to 2001 Bhuj earthquake, 1989 Loma Prieta earthquake, 1989 Loma Gilroy motion and 1995 Kobe earthquake, respectively. The Bhuj motion pertains to the devastating earthquake that struck the Kutch area in Gujarat at 8.46 AM (IST) on 26 January 2001 and had a magnitude of M_w 7.7. The epicenter of the earthquake was located at $23.4^\circ N$, $70.28^\circ E$ and at a depth of about 25 km to the north of Bacchau town. The earthquake characteristics like date of occurrence, recording station, distance from source, Maximum horizontal acceleration, Moment magnitude M_w are presented in Table 4. Strong motions records of Kobe, Loma Prieta and Loma Gilroy earthquakes are obtained using the data base available in DEEPSOIL (Hashash et al. 2008) which were effectively used earlier by Choudhury and Savoikar (2009) for equivalent-linear ground response analysis of municipal solid waste material. The earthquake characteristics of these motions like predominant Period, mean period, bracketed duration and significant duration are also presented in Table 4. These are derived using Seismo Signal program (see www.SeismoSoft.com). The strong ground motions have mean time period varying from 0.391 to 0.641 s, which represents the best frequency content characterization parameter of the ground motion. Thus the selected ground motions represent wide variation of MHA, mean time period and duration. The time history of 2001 Bhuj motion is shown in Fig. 3a (GovindaRaju et al. 2004) and the time histories of 1995 Kobe 1989 Loma Prieta, and 1989 Loma Gilroy are shown in Fig. 3b–d, respectively.

The various input parameters considered are presented in Tables 5, 6, 7, 8, 9, 10 and 11.

5 Results and Discussion

The analysis is first carried out using 2001 Bhuj earthquake (MHA = 0.106 g) as input motion (Fig. 3a). All the soil sites have been analysed using this input motion at bed rock level. The frequencies obtained for these soil sites have been compared with the fundamental frequency based on closed form solutions and are presented in Table 12. It can be observed that the results are in good agreement with the closed form solutions (Kramer 2005). Other

Table 5 Input parameters for MBH#1

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	b	D
1	Back-fill	1.5	16	203	0.5	0.0183	0.01380	1	1	0	0
2	Loose-sand	1.5	17	218	0.5	0.0326	0.02857	1	1	0	0
3	Loose-sand	1.5	17	226	0.5	0.0433	0.04000	1	1	0	0
4	Loose-sand	1.5	18	245	0.5	0.0431	0.04700	1	1	0	0
5	Clay	2.0	18	268	0.5	0.0812	0.10700	1	1	0	0
6	Clay	1.8	18	293	0.5	0.0733	0.11700	1	1	0	0

Table 6 Input parameters for MBH#2

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	B	D
1	Back fill	1.5	16	195	0.5	0.019910	0.01385	1	1	0	0
2	Loose sand	4.5	17	232	0.5	0.028838	0.02857	1	1	0	0
3	Clayey soil	2.0	17	245	0.5	0.082000	0.09046	1	1	0	0
4	Clayey soil	2.0	18	257	0.5	0.082467	0.09992	1	1	0	0

Table 7 Input parameters for WBH#1

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	B	D
1	Back fill	3.5	16	211	0.5	0.04000	0.03233	1	1	0	0
2	Silty gravel	1.5	17	263	0.5	0.04000	0.04969	1	1	0	0
3	Silty gravel	2.0	17	278	0.5	0.04000	0.05800	1	1	0	0
4	Silty gravel	3.5	18	421	0.5	0.02280	0.074500	1	1	0	0

Table 8 Input parameters for WBH#2

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	b	D
1	Back fill	1.0	16	195	0.5	0.01327	0.00923	1	1	0	0
2	Back fill	2.5	16	165	0.5	0.068274	0.03435	1	1	0	0
3	Clay with sand	2.7	17	293	0.5	0.074635	0.11761	1	1	0	0

Table 9 Input parameters for BBH#1

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	b	d
1	Filled up soil	2.0	16	126	0.5	0.06393	0.01847	1	1	0	0
2	Clayey soil	2.0	17	203	0.5	0.132595	0.100394	1	1	0	0
3	Clayey soil	1.5	17	232.5	0.5	0.107587	0.106620	1	1	0	0
4	Clayey soil	1.5	18	343	0.5	0.05233	0.112850	1	1	0	0

Table 10 Input parameters for BBH#2

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	B	d
1	Filled up soil	2.0	16	107	0.5	0.088427	0.018475	1	1	0	0
2	Clayey soil	2.0	17	186	0.5	0.158509	0.100394	1	1	0	0
3	Clayey soil	1.5	17	219	0.5	0.210244	0.106620	1	1	0	0
4	Clayey soil	1.5	18	279	0.5	0.079318	0.112850	1	1	0	0

Table 11 Input parameters for BBH#3

Layer no.	Layer	Thickness	Unit wt. (kN/m ³)	V _s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	B	d
1	Filled up soil	2.0	16	141	0.5	0.050788	0.018475	1	1	0	0
2	Clayey soil	2.0	17	176	0.5	0.176380	0.100394	1	1	0	0
3	Clayey soil	1.5	17	219	0.5	0.121707	0.106620	1	1	0	0
4	Clayey soil	1.5	18	278	0.5	0.079318	0.108855	1	1	0	0

Table 12 Comparison between the calculated frequencies and those obtained by using DEEPSOIL for various soil sites in Mumbai city

Sl. no.	Bore hole	T(sec) = $\sum 4H/V_s$ (Kramer)	f (Hz) = 1/T	T (sec) (DEEPSOIL)	f (Hz) (DEEPSOIL)	% difference
1	MBH#1	0.151	6.62	0.145	6.88	3.87
2	MBH#2	0.172	5.81	0.189	5.30	8.76
3	WBH#1	0.151	6.61	0.145	6.90	4.32
4	WBH#2	0.118	8.48	0.114	8.80	3.82
5	BBH#1	0.146	6.84	0.127	7.90	15.49
6	BBH#2	0.167	6.00	0.141	7.10	18.34
7	BBH#3	0.151	6.61	0.137	7.30	10.35

seismic input motions as detailed in Fig. 3b–d are also used and their influence on local soil layers is also discussed. These results finally help the engineers to design various earthquake resistant civil engineering structures (Choudhury et al. 2004).

5.1 Influence of Local Soil Sites on the Ground Response

Ground response is evaluated at all the layers of Mangalwadi, Walkeswar and BJ Marg sites. The results of time history of acceleration at ground surface were obtained for boreholes MBH#1, WBH#1 and BBH#1. The acceleration time history at ground level (GL) for Mangalwadi site at MBH#1

location is given in Fig. 4a for 2001 Bhuj motion. As can be seen in Fig. 3a, the input bed rock motion has a Maximum horizontal acceleration (MHA) of 0.106 g (g = acceleration due to gravity), where as the maximum output surface acceleration is observed as 0.251 g (Fig. 4a). This shows amplification of ground acceleration of about 2.37 times the MHA at bedrock level. Similarly the analysis is carried out for MBH#2 site location for 2001 Bhuj motion resulted in an amplification of 2.50 time the MHA at bedrock.

Ground response analysis is also performed for Walkeswar site WBH#1 and BJ Marg site BBH#1. It was observed that the MHA at bedrock level is amplified by about 2.62 and 3.44 and the corresponding ground accelerations are estimated as 0.278 and

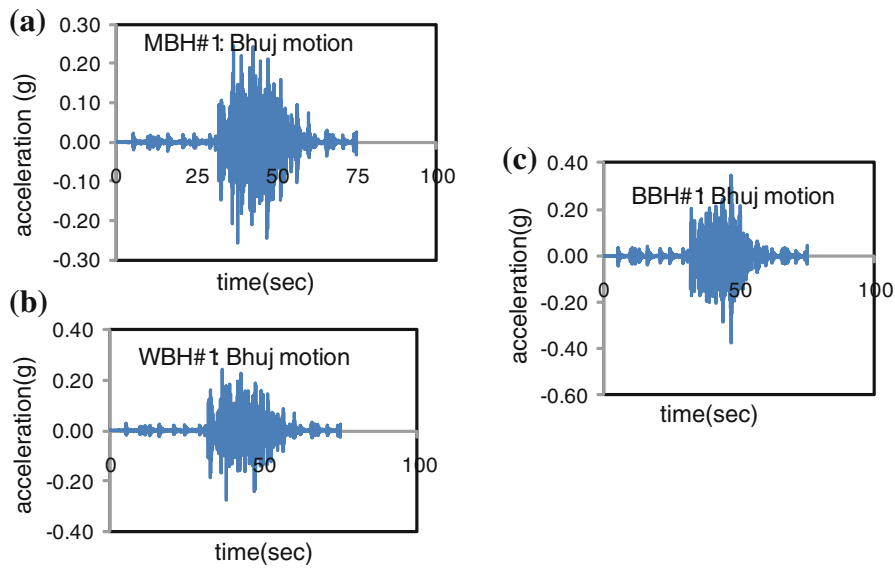


Fig. 4 **a** Acceleration time history at GL for Bhuj motion for MBH#1. **b** Acceleration time history at GL for Bhuj motion for WBH#1. **c** Acceleration time history at GL for Bhuj motion for BBH#1

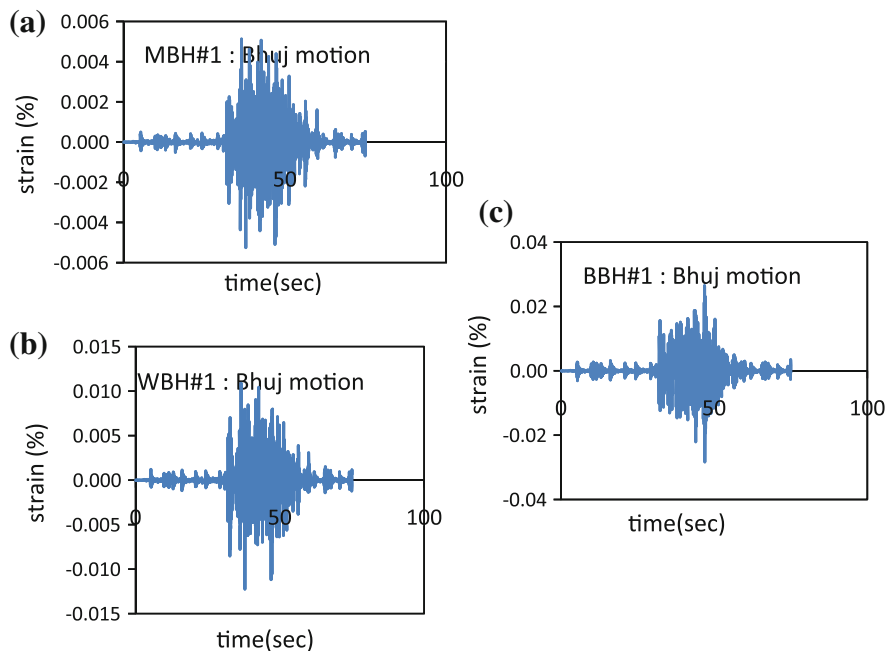


Fig. 5 **a** Strain versus time at GL for Bhuj motion for MBH#1. **b** Strain versus time at GL for Bhuj motion for WBH#1. **c** Strain versus time at GL for Bhuj motion for BBH#1

0.364 g for Walkesewar and BJ Marg sites respectively. The acceleration time histories for these sites at ground level are presented in Fig. 4b–c, respectively.

These results clearly show the effect of local soil conditions in amplifying the ground response. The studies on local site effects by Govindaraju et al.

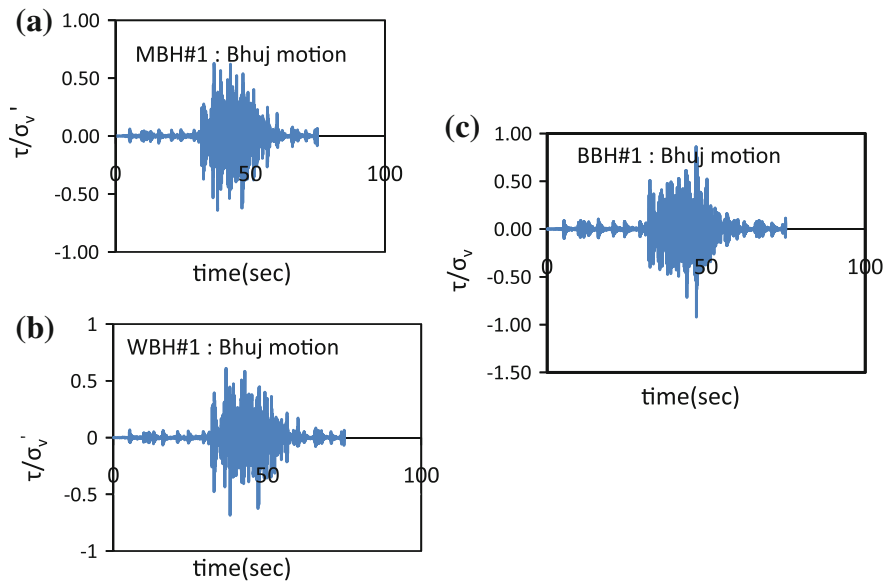


Fig. 6 **a** (τ/σ_v') versus time at GL for Bhuj motion for MBH#1. **b** (τ/σ_v') versus time at GL for Bhuj motion for WBH#1. **c** (τ/σ_v') versus time at GL for Bhuj motion for BBH#1

(2004) for a 15 m deep soil deposit near Ahmedabad city in India using SHAKE, resulted in an amplification of acceleration at the ground surface of about 1.66 to the bed rock motion.

The variation of strain levels at ground surface with time for MBH#1, WBH#1 and BBH#1 are represented in Fig. 5a–c, respectively in terms of strains versus time. From these results the degradation in soil stiffness and damping ratio of the soils corresponding to effective strains can be assessed. Also the ratio of shear stress to effective vertical stress (τ/σ_v') for these bore holes are given in Fig. 6a–c. These are useful for evaluating the liquefaction potential of the soil sites. It may be observed that the ratio of τ/σ_v' at ground surface are found to be 0.629, 0.682 and 0.91 for MBH#1, WBH#1 and BBH#1, respectively. The higher accelerations experienced in BBH#1 have resulted in higher shear stresses as can be seen from these figures.

Also the Fourier amplification ratio's have been presented in Fig. 7a–c for for these soil sites for MBH#1, WBH#1 and BBH#1, respectively. It was observed that the Fourier amplification ratio (surface/bedrock) is about 14.75, 14.70 and 25.78 for MBH#1, WBH#1 and BBH#1 soil sites, respectively. The natural frequencies of the soil deposits

can also be predicted using these results. The frequencies corresponding to peak values of Fourier amplification ratio's represent the natural frequencies of the soil deposits. The high amplifications emphasize the role of local soil sites in modifying the ground response.

5.2 Influence of Input Motion on Ground Response

The acceleration response spectrum at ground level for Mangalwadi site for different input motions are evaluated for MBH#1 site location and the results are plotted in Fig. 8a. Damping ratio is considered as 5% for generating the response spectrum. It is observed that the maximum surface acceleration for Bhuj input motion is about 1.28 g and this corresponds to time period of 0.15 s which is also the fundamental period of the soil deposit. Also it can be seen that Bhuj motion influences soil deposits of short period region. The peak spectral accelerations for Loma Prieta ground motion was found to be about 3.66 g at a time period of about 0.18 s. For Loma Gilroy motions, the maximum acceleration was found to be 3.75 g and corresponds to a time period of 0.23 s. Also it is observed that maximum spectral acceleration for

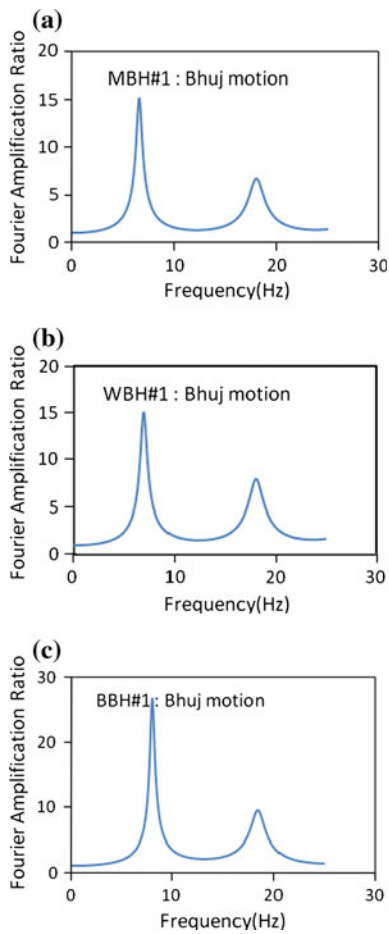


Fig. 7 **a** Fourier amplitude ratio (FAR) versus frequency at GL for Bhuj motion for MBH#1. **b** Fourier amplitude ratio (FAR) versus frequency at GL for Bhuj motion for WBH#1. **c** Fourier amplitude ratio (FAR) versus frequency at GL for Bhuj motion for BBH#1

Kobe input motion is at 0.47 s period and is found to be 2.88 g. It can be seen that Kobe earthquake is more damaging for flexible structures resting on soft soils.

Peak spectral accelerations for different seismic motions for Walkeswar site are given in Fig. 8b. The peak spectral acceleration is about 4.26 g in case of Loma Prieta motion and is observed at 0.18 s. The peak spectral accelerations for Loma Gilroy motion was found to be 3.52 g occurring at 0.23 s. For Bhuj and Kobe motions the peak acceleration are found to be 1.09 and 2.69 g, at 0.15 and 0.47 s, respectively.

The variation of period versus spectral amplification for BJ Marg site at BBH#1 location is shown in Fig. 8c. It was observed that as the natural time period

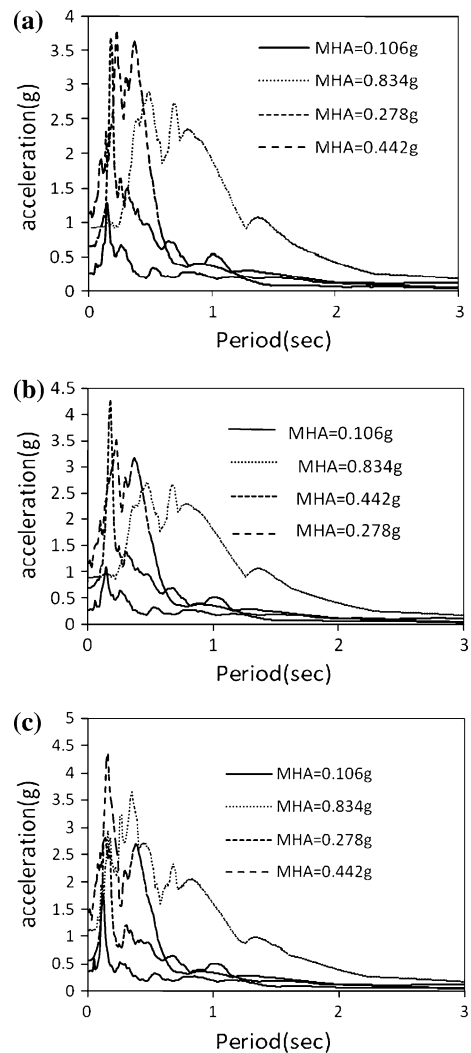


Fig. 8 **a** Response Spectrum for 5% damping at GL for MBH#1. **b** Response Spectrum for 5% damping at GL for WBH#1. **c** Response Spectrum for 5% damping at GL for BBH#1

of the soil deposit of BBH#1, is lower (0.127 s) compared to natural periods of MBH#1 and WBH#1 (0.145 s), the peak spectral accelerations have occurred earlier than the peak accelerations of MBH#1 and WBH#1 soils sites. Also it was observed that Kobe motion is amplifying the response of flexible structures resting on soft soils.

Figure 9a shows the variation of Fourier amplitude ratio versus frequency for different ground motions for MBH#1 location at ground level. It can be seen that 2001 Bhuj motion produced high amplification of about 14.75. Thus even though

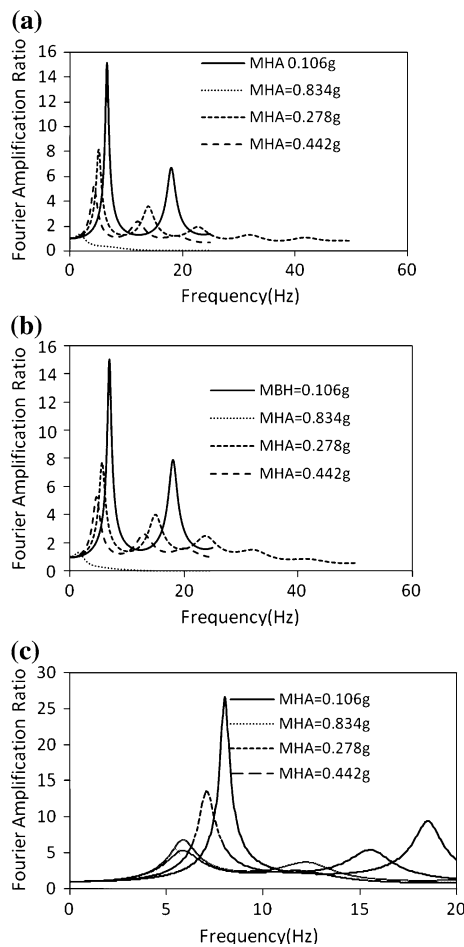


Fig. 9 **a** Fourier amplitude ratio (FAR) versus frequency (Hz) at GL for MBH#1. **b** Fourier amplitude ratio (FAR) versus frequency (Hz) at GL for WBH#1. **c** Fourier amplitude ratio (FAR) versus frequency (Hz) at GL for BBH#1

MHA of Bhuj motion is lower compared to the other seismic input motions, due to high frequency content and higher duration of the Bhuj motion, higher amplifications are observed. Also the predominant period of Bhuj motion is about 0.26 s which is closer to the fundamental period of the soil deposit (0.145 s) than other input motions as can be seen from Table 12. This shows that the amplification depends not only on the amplitude, but also on duration and frequency content of the input motion.

Fourier amplitude ratio versus frequency for different ground motions for WBH#1 and BBH#1 sites are plotted for different strong motions considered and their results are present in Fig. 9b–c, respectively. It

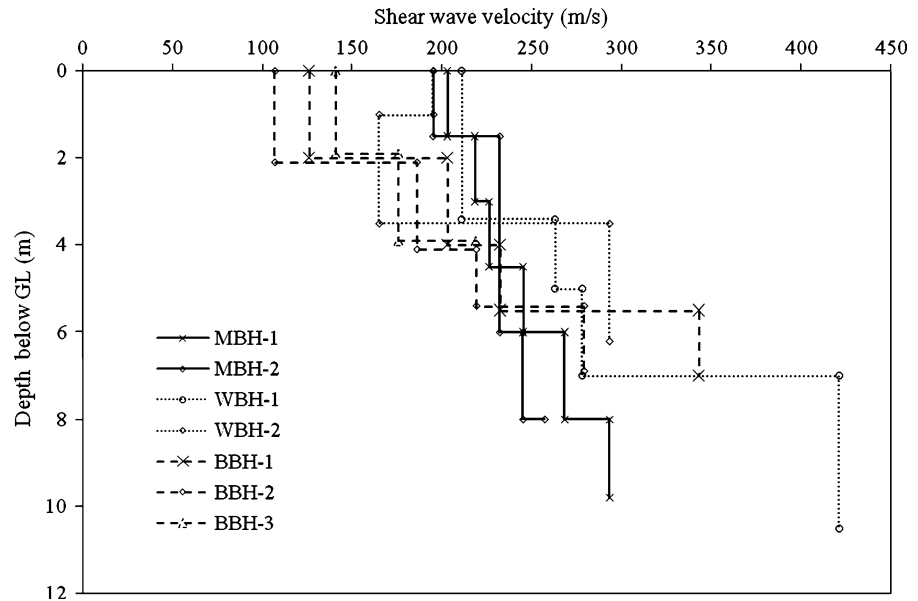
was observed that the amplifications for BBH#1 have occurred at slightly higher frequencies compared to MBH#1 and WBH#1 sites as these sites have higher natural periods than BBH#1. Again the results show Bhuj motion has higher ground amplifications. Higher amplifications have been observed for BBH#1 due to higher ground accelerations experienced by the soil site.

5.3 Influence of Soil Layers on Ground Response

The effect of soil layers on ground amplifications is also attempted in the present study. The variation of the shear wave velocity along depth for all the soil sites is represented in Fig. 10. It may be seen that the shear wave velocity is fairly constant for Mangalwadi site. Also Walkeswar site has higher shear wave velocity at bed rock level, compared to other sites. The variation of maximum horizontal acceleration along the depth is presented for Mangalwadi site MBH#1 in Fig. 11a. It observed that for Kobe motion the ground accelerations have amplified with an amplification of 1.10. The MHA obtained using Kobe motion was 0.915 g against bedrock MHA of 0.834 g. Loma Gilroy seismic motion produced higher ground acceleration amplifications with an amplification factor of 2.57. The MHA observed at ground surface using this seismic input motion (applied at bedrock location) is 1.137 g. The MHA at ground surface for 2001 Bhuj motion is found to be 0.251 g and for 1989 Loma Prieta earthquake motion it is observed as 0.64 g. The amplification factors using these input motions are 2.37 and 2.31, respectively. The variation of amplification of maximum horizontal acceleration with respect to MHA at bed rock is plotted in Fig. 11b. It was observed that though Kobe motion has high MHA, due to lower bracketed duration and low frequency content could not amplify the ground response. On the other hand though Bhuj motion has low MHA, due to high bracketed duration and high frequency content could amplify the ground response. Thus ground response depends not only on amplitude but also on mean time period, and duration of earthquake.

The variation of MHA along the depth of soil deposit is also studied for Walkeswar site and the results are presented in Fig. 11a. The amplification using Kobe input motion is observed as 1.07 with

Fig. 10 Variation of shear wave velocity with depth



MHA at ground surface equal to 0.889 g. Loma Gilroy input motion was found to produce maximum ground amplification with an amplification factor of 2.72. The amplification factors for Bhuj and Loma Prieta motions are found to be 2.62 and 2.51, respectively. Figure 11a also shows the variation of MHA along depth of the soil deposit for BJ Marg soil site BBH#1. The ground acceleration amplifications are 3.44, 3.35, 2.04 and 1.34, respectively for Bhuj, Loma Gilroy, Loma Prieta and Kobe motions, respectively. Amplification of Peak ground acceleration at surface layer with respect to Maximum horizontal acceleration at bedrock obtained by using DEEPSOIL for various soil sites in Mumbai city for different strong motion earthquakes have been summarized in Table 13.

The response spectrum variation along the soil layers is studied for MBH#1 soil site for the four strong motion records. The top layer consists of filled up material of 1.5 m thickness and has a shear wave velocity of 203 m/s. Layer 2, layer 3 and layer 4 consists of loose sand of thickness 1.5 m each and with shear wave velocities of 218, 226 and 245 m/s, respectively. Layer 5 and layer 6 are clay layers with thickness of 2.0 m and 1.8 m with shear wave velocities of 268 m/s and 293 m/s respectively. The response spectrum results for Bhuj motion are given in Fig. 12a. It can be seen that the soil layers have

profound influence in modifying the ground response. Also the results for other seismic input motions viz., Loma Prieta motion, Loma Gilroy motion and Kobe motion are presented in Fig. 12b–d, respectively. This study is very useful for designers as response spectrum is available at different founding depths. It can be seen that 1995 Kobe motion has less influence on the local soil site ground response than other motions, for short period region. Also the variation of Fourier amplitude ratio versus frequency along soil layers considering the four different ground motions are plotted in Fig. 13a–d for Bhuj, Loma Prieta, Loma Gilroy and Kobe motions, respectively. It can also be seen that 2001 Bhuj motion has amplified the soil layers more than other motions and also amplification due to Kobe motion has less influence on the soil layers compared to the other motions.

6 Conclusions

A case study on the ground response analysis of some typical Mumbai soil sites considering four different strong motion earthquakes is presented. The following conclusions are obtained.

- The role of local soil sites in amplification of responses has found to be significant and has been

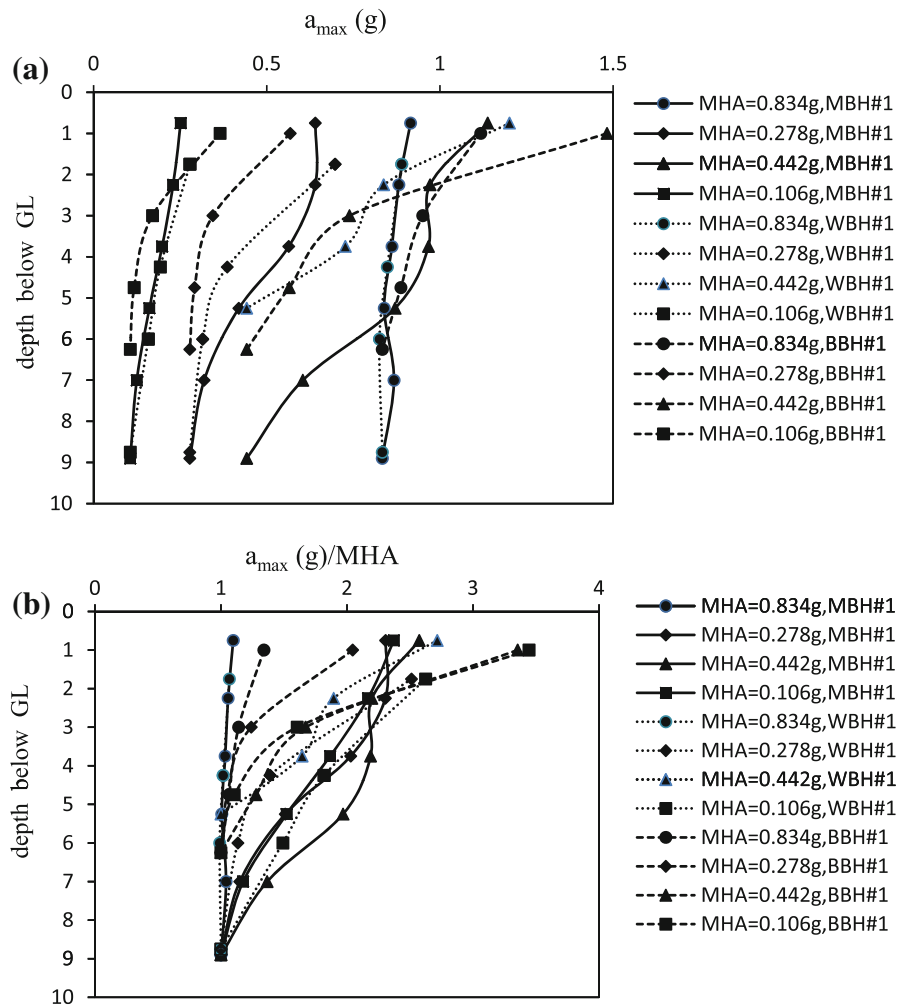


Fig. 11 **a** Maximum acceleration $a_{max}(g)$ versus depth (m). **b** Amplification of acceleration versus depth (m)

Table 13 Amplification of peak ground acceleration at surface layer with respect to maximum horizontal acceleration (MHA) at bedrock obtained by using DEEPSOIL program for various soil sites in Mumbai city for different strong motion earthquakes

S.no	Bore hole	Input motion	Ground amplification ($MHA_{surface}/MHA_{bedrock}$)
1	MBH#1	Bhuj 2001 (MHA = 0.106 g)	2.37
		Loma Prieta 1989 (MHA = 0.278 g)	2.31
		Loma Gilroy 1989 (MHA = 0.442 g)	2.57
		Kobe 1995 (MHA = 0.834 g)	1.10
2	WBH#1	Bhuj 2001 (MHA = 0.106 g)	2.62
		Loma Prieta 1989 (MHA = 0.278 g)	2.51
		Loma Gilroy 1989 (MHA = 0.442 g)	2.72
		Kobe 1995 (MHA = 0.834 g)	1.07
3	BBH#1	Bhuj 2001 (MHA = 0.106 g)	3.44
		Loma Prieta 1989 (MHA = 0.278 g)	2.04
		Loma Gilroy 1989 (MHA = 0.442 g)	3.35
		Kobe 1995 (MHA = 0.834 g)	1.34

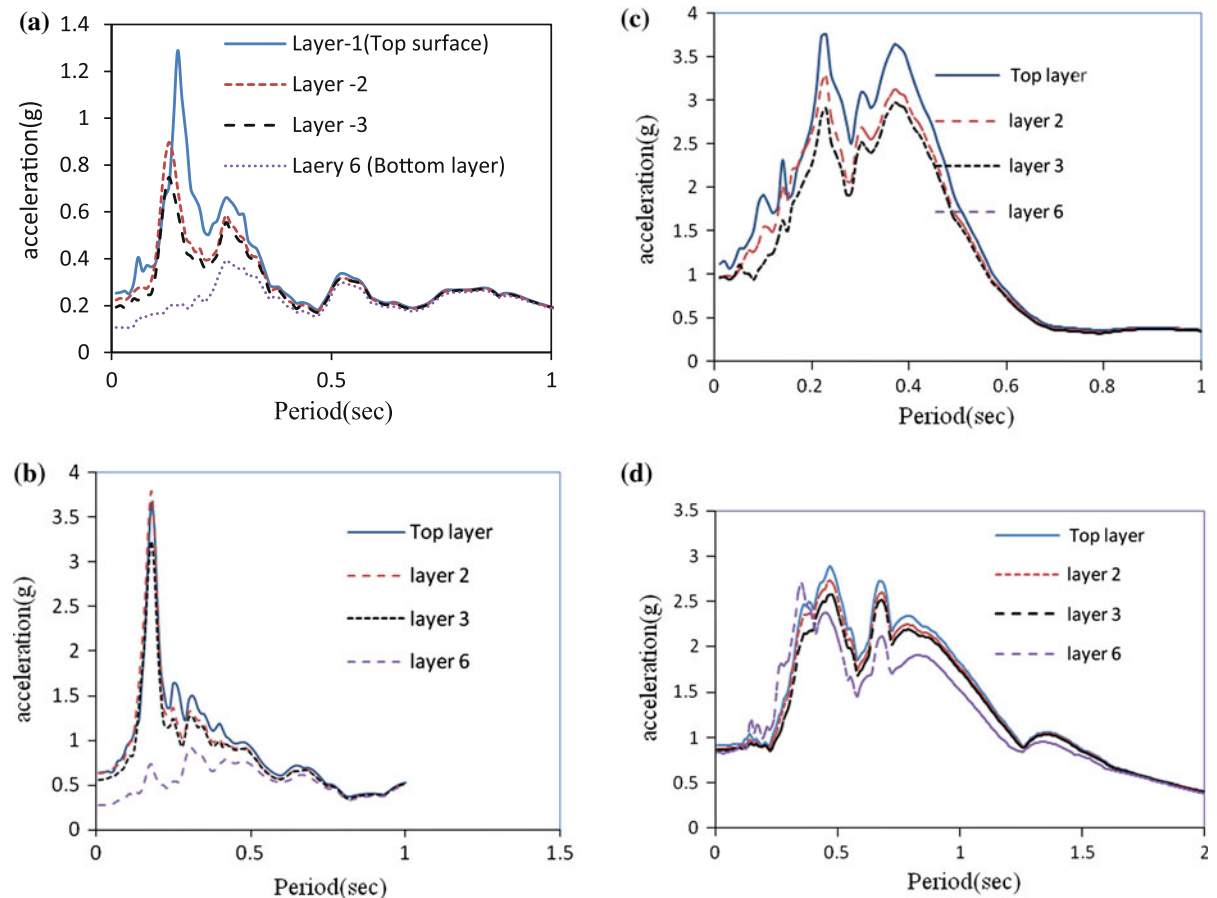


Fig. 12 a Response spectra of soil layers along depth (MHA = 0.106 g, MBH#1, damping = 5%). **b** Response spectra of soil layers along depth (MHA = 0.278 g, MBH#1, damping = 5%). **c** Response spectra of soil layers along depth

(MHA = 0.442 g, MBH#1, damping = 5%). **d** Response spectra of soil layers along depth (MHA = 0.834 g, MBH#1, damping = 5%)

discussed thoroughly. It is observed that local soil sites have a profound influence in modifying the ground response.

- The peak ground acceleration amplification factors for 2001, Bhuj input motion are found to be about 2.50 for Mangalwadi site, 2.60 for Walkeswar site and 3.45 for BJ Marg site. Peak ground amplifications for different earthquake motions have also been evaluated and can readily be used by designers.
- Natural frequencies of the soil sites have also been evaluated by using both analytical formula and

DEEPSOIL program and found to match well which in turn shows the validation of DEEPSOIL program for such problem. Response spectra considering 5% damping and Fourier amplitude ratio's have been obtained for these soil sites with different input motions.

- The response spectrum along various soil layers for four strong motion earthquakes with wide variation from low to high MHA and mean time periods are obtained. These results may be used alternatively in the absence of any site specific data for similar sites of Mumbai region.

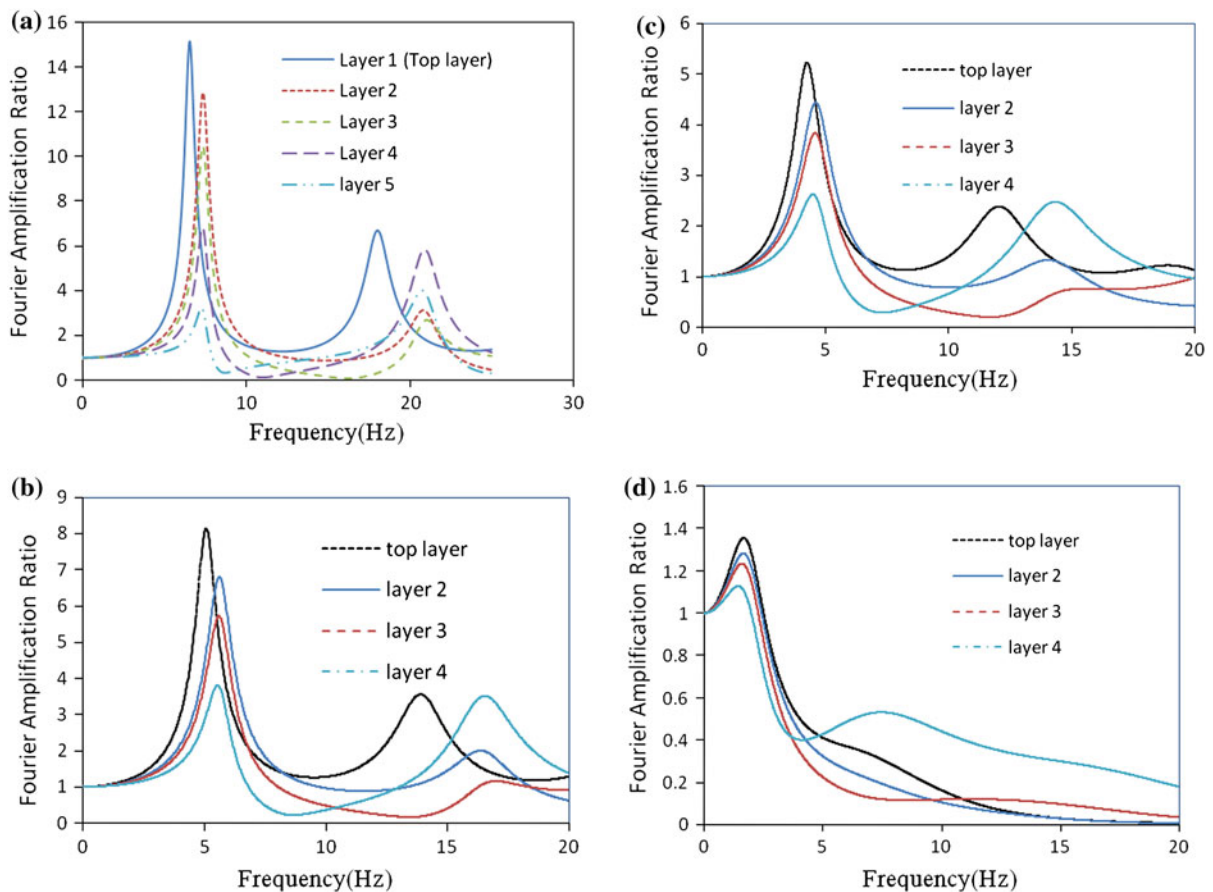


Fig. 13 **a** Fourier amplitude ratio (F.A.R.) versus frequency along depth of soil deposit (MHA = 0.106 g, MBH#1, damping = 5%). **b** Fourier amplitude ratio (F.A.R.) versus frequency along depth of soil deposit (MHA = 0.278 g, MBH#1, damping = 5%). **c** Fourier

amplitude ratio (F.A.R.) versus frequency along depth of soil deposit (MHA = 0.442 g, MBH#1, damping = 5%). **d** Fourier amplitude ratio (F.A.R.) versus frequency along depth of soil deposit (MHA = 0.834 g, MBH#1, damping = 5%)

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