

Impact of Ground Response Analysis on Seismic Behavior and Design of Piles in Kolkata City

Kaustav Chatterjee¹

Received: 30 April 2017 / Accepted: 24 August 2017 / Published online: 7 September 2017
© Indian Geotechnical Society 2017

Abstract Kolkata city is a major metro city of eastern India and spread in a north–south direction along the east bank of the Hoogly river with the soil being predominantly soft, thick and alluvial in origin. According to seismic code of India, Kolkata city is located along the boundary of seismic zone III and IV which indicates moderate to high seismic hazard. In this paper, using derived empirical correlations between shear wave velocity V_s and SPT N values, soil site classification is carried out and it indicated CLASS E category, highlighting the seismic hazard of Kolkata city. The alluvial nature of soil located in Kolkata city is prone to soil amplification when subjected to different earthquake motions at the bedrock level and amplification factor for bedrock level acceleration varying between 1.7 and 2.5 is obtained in the present study. The spectral acceleration at 5% damping ratio is observed to be 1.94 g, when subjected to 1995 Kobe earthquake motion. The results obtained from ground response analysis is used for seismic design of pile foundations passing through liquefiable and non-liquefiable soil strata by considering both kinematic and inertial loading. The maximum bending moment is observed to occur at the interface of liquefiable and non-liquefiable soil layers. Both bending moment and pile head deflection showed a substantial increase in magnitude when inertial loads are considered in the analysis in addition to kinematic loading. Hence the present study illustrates the significance of deflection and bending response of pile foundations in liquefiable soil as important

parameters for seismic design of pile foundations in Kolkata city.

Keywords Liquefaction · Shear wave velocity · Soil amplification · Pile · Kolkata · Earthquake

Introduction

Hazard assessment is conducted to estimate quantitatively and define the hazard expected in a particular area due to the strategic location of the place near active faults and lineaments and earthquake events that have occurred in the past and have a probability of taking place in the future. Hazard assessment is generally conducted using a deterministic approach or probabilistic approach. In deterministic seismic hazard analysis (DSHA) the earthquake source causing the highest hazard at a particular location is considered to be the causative source and using attenuation relationships, the hazard level at that site is determined [1]. However in case of probabilistic seismic hazard analysis (PSHA) the uncertainties associated with the size of an earthquake, the location of occurrence and other factors are considered. The critical part of seismic hazard assessment is the determination of maximum horizontal acceleration (MHA) at ground surface (also called peak ground acceleration (PGA)) and spectral acceleration (S_a) of the particular area. The results of such hazard studies are available as seismic hazard maps showing the contours of peak ground acceleration across the area which are essential parameters for earthquake resistant design of structures in seismic prone regions.

It is a challenging task for civil engineers to provide safe and economic design of multi-storied buildings and high rise constructions in urban cities like Kolkata, where the

✉ Kaustav Chatterjee
kchatf@iitr.ac.in

¹ Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee 247667, India

increasing population and accelerating infrastructural growth makes it a necessity. Although Indian standard code on earthquake resistant design of structures IS 1893: Part 1 [2] provides various guidelines for design of foundations (pile foundation, raft foundation, isolated footings) in seismic regions, the influence of various factors like local soil conditions, liquefaction hazard, earthquake parameters and ground response analysis on design of foundations needs considerable attention. Seismic hazard assessment and ground response analyses of various Indian cities have been implemented by several researchers in the past and have observed that the seismic hazard at a particular site is influenced by the topography of the bedrock, local soil conditions existing at the site and duration of shaking of the earthquake motion. These areas for which seismic hazard analysis have been implemented in the past includes Bangalore [3], Chennai [4], Delhi [5], Goa [1], Gujarat [6–8], Guwahati [9], Kolkata [10, 11], Kanpur [12], Mumbai [13–15] and others. The successful implementation of seismic ground response analysis of soil deposits at a particular location provides a geotechnical engineer with important information on natural period of soil deposits at the site, amplification of acceleration at the ground surface and response spectrum curves which are useful parameters for seismic design of pile foundations.

Various case histories accounting the failure of pile foundation due to liquefaction by treating the pile as beam elements and considering lateral loads to cause bending failure in piles were reported by Hamada [16], Ishihara [17], Tokimatsu et al. [18] and Finn et al. [19]. However piles passing through liquefiable soils are subjected to both axial and lateral loads during an earthquake and bending–buckling interaction causes vulnerable failure of pile foundation due to significant degradation of stiffness [20–23]. Several researchers have carried out seismic analysis of piles considering both kinematic and inertial loadings using experimental [24–26], analytical and numerical techniques [27–39]. The results obtained from these studies highlighted the considerable need of accurate soil–pile interaction to evaluate the response of piles passing through liquefiable soils in terms of bending moment and deflection under seismic conditions.

However, a thorough procedure on seismic analysis and design of single piles and pile group considering the influence of various parameters like local soil conditions, input ground motions, liquefaction hazards, bending–buckling interaction due to the application of vertical and lateral load on pile foundation for a metropolitan and important city like Kolkata is scarce in literature. Hence, in the present study, empirical correlations between shear wave velocity and field SPT N values have been used to implement soil site characterization of Kolkata city as per NEHRP [40] guidelines and the seismic hazard of Kolkata

city is highlighted. Kolkata city is located along the boundary of seismic zone III and IV indicating moderate to high seismic hazard and spread in a north–south direction along the east bank of the Hoogly river with the soil being predominantly soft, thick and alluvial in origin. The alluvial nature of soil located in Kolkata city is prone to soil amplification when subjected to different earthquake motions at the bedrock level, which is obtained from site specific seismic equivalent linear ground response analysis. The results obtained from seismic ground response analysis in the form of response spectrum curves and depth-wise variation of acceleration is utilized in the present study for seismic design of single piles in liquefiable and non-liquefiable soils of Kolkata city, by considering both kinematic and inertial interactions. The presence of vertical loads in addition to input ground motions is considered in the present study and bending–buckling interactions are simulated and the seismic response of pile foundation in terms of pile bending moment and pile deflection are obtained.

Kolkata City: Study Area and Need for Present Study

Kolkata city, located between latitudes $22^{\circ}18'N$ – $22^{\circ}50'N$ and longitudes $88^{\circ}08'E$ – $88^{\circ}32'E$ and covering an area of 185 km^2 , is one of the largest cities in the world. Located in eastern India, Kolkata is the capital of the Indian state of West Bengal and spread north–south along the east bank of River Hoogly. Kolkata city is a major metro city in India and serves as a gateway to north-east India. Kolkata city is located in the lower delta of River Ganga in eastern India and is also low-lying, having an elevation of 9 m above mean sea level [41]. The soil profile existing in Kolkata city is mostly soft, thick and alluvial in nature and underlain by clay, silt and silty sand sediments, having varying grain size and textures. Moreover a major part of the city, which was initially a wetland, is reclaimed by filling with silty soil, dredged from the River Ganges [10]. Due to immense infrastructural growth, rapid urbanization, increasing population and scarcity of vacant lands, multi-storied buildings founded on pile foundations are coming up in Kolkata city. Kolkata city is located at the boundary of seismic zones III and IV as per seismic code of India [2] indicating moderate to high seismic alert. The city is also situated along the Eocene Hinge Zone, a prominent regional basement fault and is surrounded by several other active faults like Jangipur–Gaibandha Fault, Pingla Fault, Rajmahal Fault, Sylhet Fault, Sainthia Bahmani Fault and Dhubri Fault [41, 42]. These are ample proofs of seismic vulnerability and hazard associated with Kolkata city and the destructions that might occur if an earthquake occurs in the future.

Kolkata city and its surrounding areas have experienced tremendous vibrations due to various earthquakes in the past. This includes the 1906 Calcutta earthquake and the Calcutta earthquake of 1964 which caused major damages to the city. In the recent years, tremors have been felt across Kolkata city due to various far source earthquakes like 2011 Sikkim, 2012 Indonesia, April 2015 Nepal and 2016 Myanmar earthquakes. This is due to alluvial nature of soil existing in Kolkata city and the significant role played by the local soils in altering the ground motion characteristics of the earthquake. Similar scenarios were also observed in San Francisco bay area during 1989 Loma Prieta earthquake and in Ahmedabad city of Gujarat, India during 2001 Bhuj earthquake. Hence in the present study, Kolkata city is chosen as the study area and using derived V_s -SPT N correlations ground response analysis is carried out which is further used for seismic analysis and design of pile foundations in liquefiable soils.

V_s - N Correlations for Kolkata city

The response of soil at a particular site under the influence of dynamic loadings is governed by damping properties, shear wave velocity and shear modulus. Although shear wave velocity can be measured at site using various field tests like seismic reflection test, seismic refraction test, Multichannel Analysis of Surface Wave (MASW) test; however in an urban city like Kolkata in situ measurement of shear wave velocity is not conducted due to lack of required free space. Hence in the absence of sophisticated dynamic field test data, empirical correlations derived between SPT N values and shear wave velocity V_s for Kolkata city [41] are used in the present study. Extensive geotechnical data from over 450 boreholes in entire Kolkata city were collected from various soil investigation agencies, government and private institutions and consultants and national labs. The correlations were derived using power model of non-linear regression analysis for various soils existing in Kolkata city of eastern India, i.e., all soils, clay, silt and silty sand and are tabulated in Table 1 [41]. The values of the various statistical parameters like coefficient of correlation (r), coefficient of regression (R^2) and residual mean square error (MS_E) are also tabulated to

highlight the accuracy of the results obtained using these empirical correlations. It is observed from Table 1 that the derived correlations have regression coefficients greater than 0.95 and low values of residual mean square error, thereby indicating an accurate prediction of shear wave velocity from SPT N values.

The proposed correlations are compared with the existing correlations for other Indian cities like Delhi [8], Bangalore [43], Chennai [44] and Mumbai [45] and illustrated in Fig. 1. It is observed from Fig. 1 that the shear wave velocity values computed for Kolkata city using the proposed V_s - N correlations are following the similar nature of curves as observed for Bangalore, Chennai, Delhi and Mumbai cities and hence compared reasonably well with the works of previous researchers. Moreover the present curve is observed to coincide with that obtained for Mumbai city due to similar soft soil profiles existing at various locations in both the cities. Further, the results calculated from the present derived correlations for clay, silt and silty sand soils of Kolkata city are compared with the correlations proposed by previous researchers across the world and shown in Fig. 2. It is seen from Fig. 2 that the calculated values of shear wave velocity for clay, silt and silty sand soils of Kolkata city are similar to that calculated using the regression equations given by other researchers. The variation in results may be attributed to the different geotechnical conditions of the study area, variability of SPT equipments, variation in level of ground water table and other similar conditions which may have a profound influence on the results.

The V_s - N correlations proposed for all soils have further been used to calculate the magnitudes of $V_{s,30}$ which is defined as the magnitude of shear wave velocity average over the top 30 m of soil deposits and used for classifying soil sites at the particular location for geotechnical earthquake engineering design problems [40]. The magnitude of $V_{s,30}$ is calculated according to the following expression:

$$V_{s,30} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{s,i}}} = \frac{30}{\sum_{i=1}^n \frac{d_i}{V_{s,i}}} \tag{1}$$

Table 1 Proposed correlations between shear wave velocity (V_s) and uncorrected SPT N values (modified after Chatterjee and Choudhury [41])

Soil type	Proposed correlation	Coefficient of regression (R^2)	Coefficient of correlation (r)	Residual mean square error (MS_E)
All soils	$V_s = 78.21 N^{0.38}$	0.96616	0.95418	0.000415
Clay	$V_s = 77.11 N^{0.39}$	0.98454	0.96055	0.000432
Silt	$V_s = 58.02 N^{0.46}$	0.97423	0.96612	0.000557
Silty Sand	$V_s = 54.82 N^{0.53}$	0.97488	0.98046	0.001520

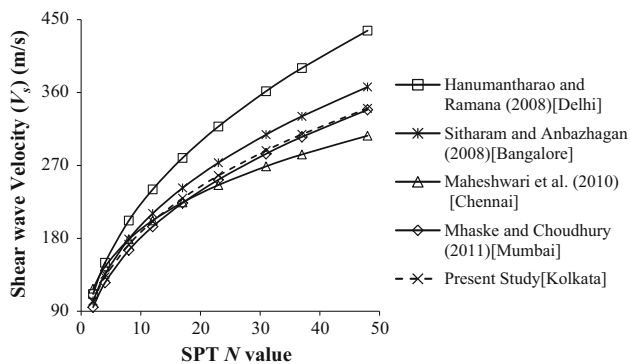
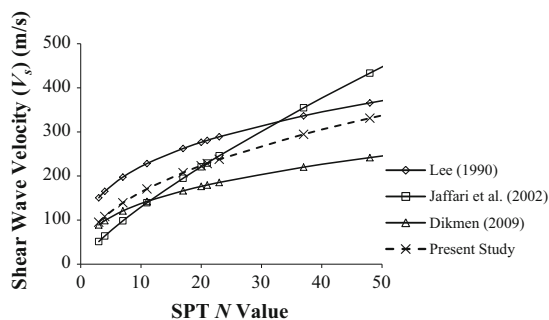
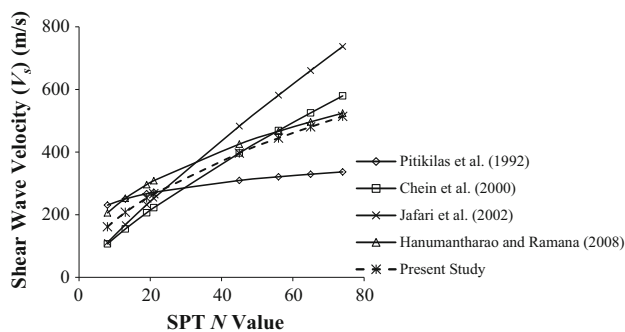


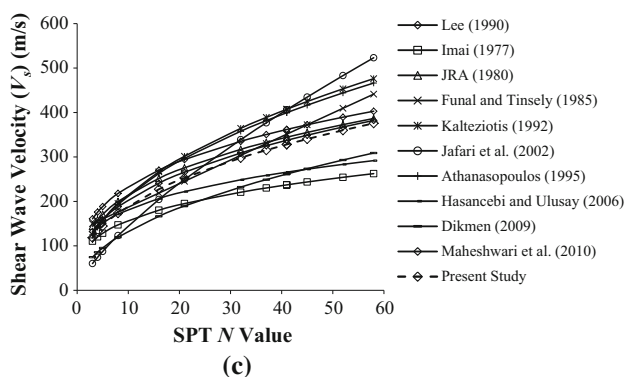
Fig. 1 Comparison of shear wave velocity for soils obtained in the present study with that of different Indian cities



(a)



(b)



(c)

Fig. 2 Comparisons between proposed shear wave velocity V_s –SPT N correlations for **a** silt **b** silty sand and **c** clay in Kolkata city and other similar relations available worldwide

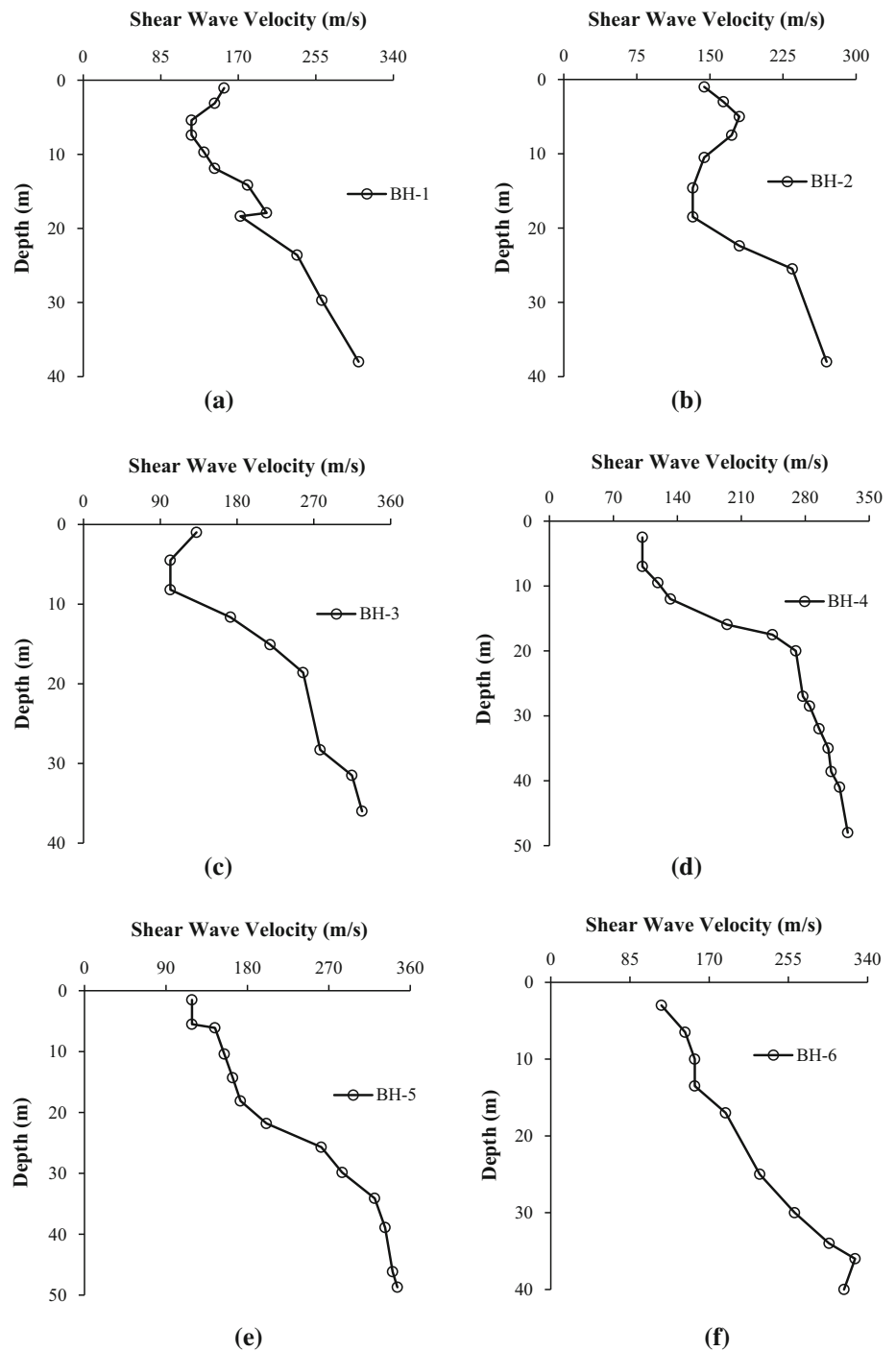
where, $\sum_{i=1}^n d_i$ is equal to 30 m, d_i denotes the thickness (in metres) and $V_{s,i}$ denotes the shear wave velocity (in m/s) of the i th soil layer respectively, existing in the top 30 m, in a total of n soil layers [41]. The variation of shear wave velocity with depth at various borehole locations is shown in Fig. 3 and the magnitude of $V_{s,30}$ at each of these locations is tabulated in Table 2. It is seen from Table 2 that the magnitude of $V_{s,30}$ calculated in these soil sites lies in the range of 165 m/s to 178 m/s, which is less than 180 m/s and thereby classifying these typical Kolkata soil sites in Class E category as per NEHRP [40] guidelines. Hence it can be inferred that several locations in Kolkata city are prone to seismic hazard at different earthquake magnitudes. Moreover the proposed correlations are further utilized for carrying out seismic equivalent linear ground response analysis of various soil sites in Kolkata city and the results obtained in the form of depth-wise variation of maximum horizontal acceleration and response spectrum curves are used for seismic design of pile foundations in Kolkata city.

Site-Specific Seismic Equivalent Linear Ground Response Analysis for Kolkata City

The passage of seismic waves from bedrock to the ground surface causes amplification of ground motion over the soil sediments. This happens because the seismic waves gets trapped in the soil sediments, resulting in varying impedance between the underlying bedrock and the soil particles [46]. As a result the amplitude and frequency content of the seismic waves are considerably modified when it travels through the soil deposits and this process where local soil layers modifies the strong motion characteristics of an earthquake is soil amplification [11, 47]. Kolkata city has soft alluvial soil due to its location along the banks of river Hoogly and hence equivalent linear ground response analysis of earthquake motions at the bedrock level is necessary and conducted for Kolkata city.

In the present study seismic equivalent linear ground response analysis is carried out at six different locations of Kolkata city, India as illustrated in Fig. 4. The boreholes are selected in such a manner such that they represent a spatial variation of soil existing at various locations across the city. The soil profiles existing at the six different locations (BH #1–BH #6) of Kolkata city are tabulated in Table 3. As observed from Table 3, the top layers of the typical soil strata BH #1 comprised of loose sand and silty clay underlain by grey silty clay and medium fine sand with grey silty clay at the bottom layers. The soil site (BH #1) is subjected to 5 different input earthquake motions, viz, 1989 Loma Gilroy, 1994 Northridge, 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions, having a wide variation of strong

Fig. 3 Typical variations of shear wave velocity V_s with depth at various locations of Kolkata city



motion parameters like frequency content, bedrock level acceleration and duration of shaking [11, 36] and the analysis is conducted using SHAKE 2000 [48] computer program. The selection of input motions, having different ground motion parameters, are done in such a manner to study its effects at different locations of Kolkata city and the destruction that can occur if any earthquake having such properties strike the city in future. The input motions are applied at the rigid bedrock level and the soil layers

were considered horizontal with the ground surface being assumed to be level [11].

The variation of acceleration with time at the ground surface due to 1995 Kobe and 2001 Bhuj earthquake motions is illustrated in Fig. 5. It is observed that the peak ground acceleration at the ground surface is 1.57 g when subjected to 1995 Kobe motion and 0.24 g when subjected to 2001 Bhuj motion. This indicates that the input acceleration is amplified by 1.88 and 2.28 times when subjected

Table 2 Average shear wave velocity till 30 m depth ($V_{s,30}$) of all soils at particular locations of Kolkata city

Location	Latitude (°N)	Longitude (°E)	Depth of borehole (m)	$V_{s,30}$ (m/s)	Site class as per NEHRP (2003)
BH-1	22.5109	88.2160	38	176.68	Class E
BH-2	22.4833	88.3055	38	169.02	Class E
BH-3	22.5621	88.4051	36	178.54	Class E
BH-4	22.5308	88.3949	48	166.50	Class E
BH-5	22.6058	88.4728	48	173.90	Class E
BH-6	22.5480	88.3518	40	178.30	Class E

to 1995 Kobe and 2001 Bhuj seismic motions, respectively. In Table 4 the dynamic soil properties at different layers of the soil column in BH #1 is calculated. The average shear wave velocity is computed to be 187.1 m/s while the fundamental time period is determined to be 0.81 s. The acceleration response spectrum at the ground surface for BH #1 due to the input ground motions, considering 5% damping ratio, and its comparison with IS 1893: Part 1 [49] is illustrated in Fig. 6. It is observed that the maximum spectral acceleration recorded is 1.94 g at a time period of 0.86 s, which is close to the fundamental time period of the soil column, i.e., 0.81 s, when subjected to 1995 Kobe motion. The peak spectral acceleration for 1989 Loma Gilroy, 1994 Northridge, 2001 Bhuj and 2011 Sikkim earthquake motions are 0.69 g at 0.42 s, 1.14 g at 0.3 s, 0.46 g at 0.26 s and 0.75 g at 0.21 s, respectively. This clearly indicates that while 1995 Kobe motion is having a detrimental influence over long, flexible structures resting on soft soils and having a long fundamental period, 2011 Sikkim motion is vulnerable for small structures having a short fundamental time period and resting on soft soils. Figure 7 shows the variations of Fourier amplification ratio with frequency for BH #1. It is observed that 2001 Bhuj and 2001 Sikkim motions produced the maximum amplifications of 4.26 and 5.34 at a frequency of 1.125 Hz and 1.25 Hz, respectively. The minimum amplification recorded is 3.82 at a frequency of 1 Hz due to 1995 Kobe motion. This is due to the higher duration and frequency content of 2001 Bhuj and 2011 Sikkim seismic motions resulting in higher amplification as compared to 1995 Kobe motion which, in spite of having a higher acceleration at bedrock level (0.834 g), do not amplify significantly due to its lower duration. The depth-wise variation of maximum horizontal acceleration (MHA) and the amplification factor is illustrated in Fig. 8 and tabulated in Table 5. It is observed for 1995 Kobe motion the MHA at the ground surface (which is also called the peak ground acceleration (PGA)) is 1.57 g against the bedrock level acceleration (a_{max}) of 0.834 g; thereby indicating an amplification factor of 1.88. Similarly for 1989 Loma Gilroy, 1994 Northridge, 2001 Bhuj and 2011 Sikkim motions the

corresponding PGA are observed to be 0.63 g, 1.08 g, 0.24 g and 0.51 g, respectively. The corresponding amplification factors for the same sequence of motions are calculated to be 1.69, 1.90, 2.28 and 2.52, respectively. Thus it can be inferred that the amplification of acceleration is significantly affected by the frequency content and duration of the input seismic motions and the bedrock level acceleration has little influence over the same. Moreover, due to soft alluvial and clay soil layers existing in Kolkata city, high values of amplification of bedrock motion, varying between 1.6 and 2.5, are obtained in the present study. As a result soft soil sites, which undergoes large amplification, should be given proper attention for seismic analysis and design of pile foundations in Kolkata city.

Seismic Analysis of Piles in Liquefiable Soil

The analysis of piles passing through liquefiable and non-liquefiable soils and subjected to earthquake motions is an interesting area of research and that too for a densely populated city like Kolkata where most of the buildings are constructed on pile foundations. The soil site characterization as per NEHRP [40] and amplification of bedrock level acceleration at the ground surface is an indication of the seismic risk the city is exposed to. Although most of the existing methods of seismic analysis of pile foundations consider the effect of bending failure due to lateral loads and buckling failure due to vertical loads separately; however an accurate design procedure should consider bending-buckling interaction mechanism of pile failure in liquefiable soil. In the present study a vertical concrete pile having length l , diameter d and flexural stiffness $E_p I_p$ is embedded in a two-layered soil medium comprising of a liquefiable sand layer of thickness L_{liq} underlain by a non-liquefiable stiff clay layer of thickness L_{nliq} , as shown in Fig. 9. The pile is subjected to a vertical compressive load having magnitude P applied at the centre of the pile section on the ground surface. In addition to it, the pile tip is also subjected to a dynamic loading generated due to an input ground motion and horizontal lateral loads at various nodes along the pile depth which is calculated according to

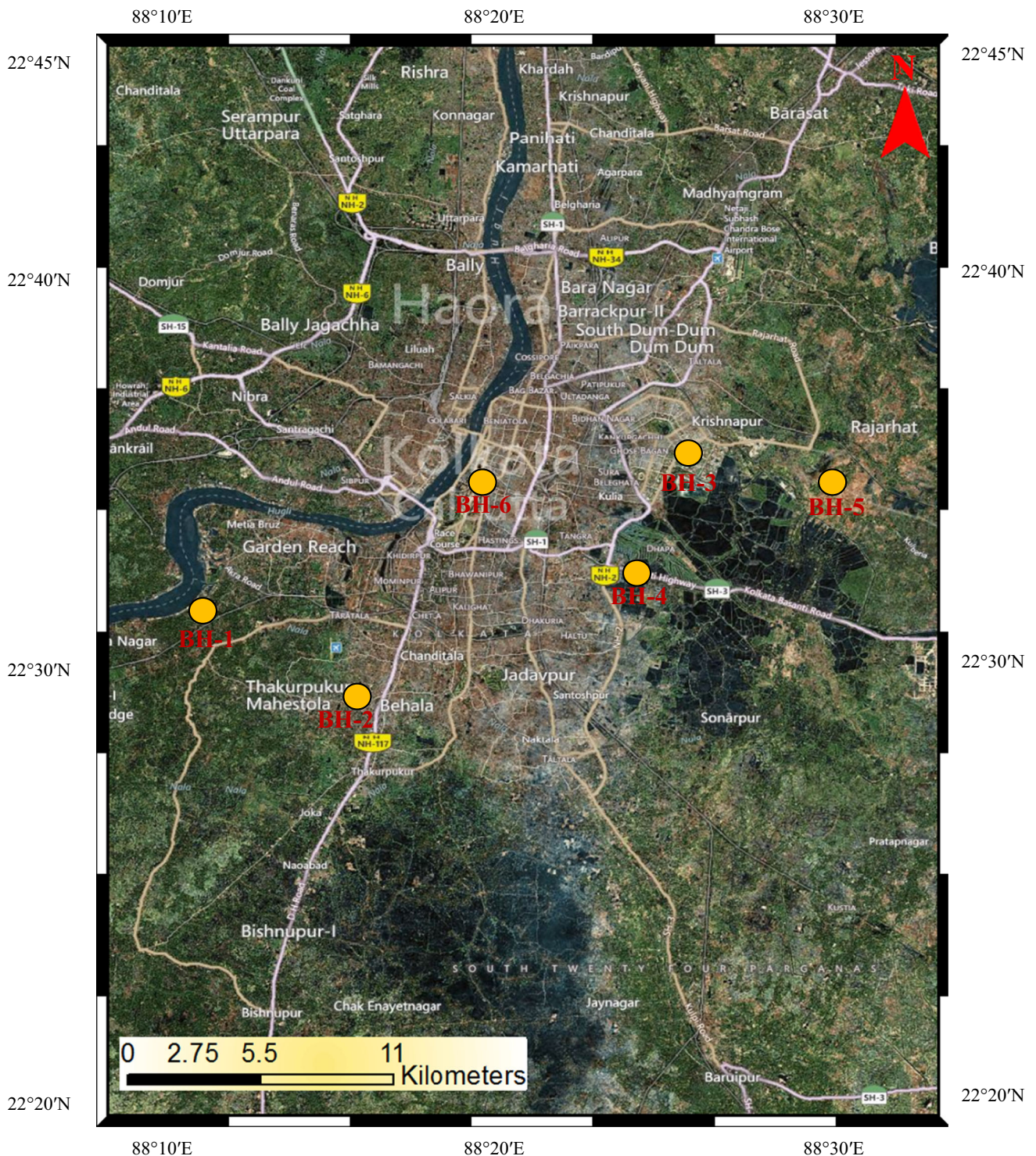


Fig. 4 Map of Kolkata city showing the location of the six boreholes considered in the present study

Chatterjee et al. [36]. The pile will experience lateral deflection dependent upon the magnitude and direction of the applied loading. The lateral resistance developed by the pile when subjected to combined loading is analyzed using the modulus of subgrade reaction approach, which is

considered to be dependent on depth below the ground surface. Hence the governing differential equation for determining the horizontal deflection of the pile (y) along the depth (z) when subjected to the above mentioned combined loadings is given as [50, 51]:

Table 3 Soil profiles of BH #1–BH #6 considered in the present study

Borehole no.	GWT (m)	Depth (m)	Soil type	SPT <i>N</i>	Unit weight [γ] (kN/m ³)
BH #1	2.4	1.05	Brownish grey silty clay with organic matters	6	17.6
		7.4	Brownish grey silty fine sand with organic matters	3	16.3
		11.9	Deep grey silty clay with decomposed wood and organic matters	5	17.8
		14.15	Brownish grey silty fine sand with traces of mica	10	18.7
		18.4	Yellowish brown silty sand with mica	8	19.6
		23.6	Brownish grey silty clay with kankars	18	20.7
		30.4	Steel grey silty fine sand with mica	24	19.8
		38	Brownish grey silty clay with kankars and wood	35	20
BH #2	2.1	1	Brownish grey clayey silt with kankars	5	14.7
		5	Blackish peat with decomposed wood	9	15.8
		10.5	Deep grey clayey silt with decomposed organic matters	5	17.7
		18.5	Steel grey silty clay with organic matters	4	18
		25.5	Deep grey silty clay with fine sand with nodules, rusty spots	18	18.2
		38	Deep grey silty clay with organic matters	26	18.7
BH #3	1.6	4.5	Deep grey clayey sandy silt/clayey silty sand	2	17.4
		11.65	Bluish grey silty clay with yellowish spots	8	17.1
		15.1	Brownish grey silty fine sand with steel grey spots with clay binders	15	17.8
		18.6	Brownish grey silty fine sand	23	18
		28.3	Brownish grey silty clay with calcareous nodules and rusty spots	28	18
		36	Brownish grey silty clay with rusty spots.	43	18.2
BH #4	1.9	2.5	Very soft, light blackish grey, silty clay	2	14.6
		9.5	Very soft, light blackish grey to deep greyish black, clayey silt	3	14.9
		16	Medium, light blackish grey, silty clay.	11	16.1
		27	Hard, yellowish grey, clayey silt with brownish patches	28	17
		32	Dense to very dense, light yellowish grey, silty sand with mica	33	17.4
		48	Hard, steel grey to deep yellowish grey, silty clay	43	17.9
BH #5	1.65	1.5	Blackish grey rubbish, brickbats, etc. (Filled up soil)	3	15.8
		6.1	Blackish grey decomposed and semi decomposed rubbish	5	16
		18.1	Deep grey silty clay with decomposed wood and organic matters	8	16.5
		25.7	Yellowish grey clayey silt with steel grey spots	24	17
		34.1	Brownish grey clayey silt with steel grey patches	41	17.4
		48	Greyish yellowish grey silty clay with steel grey spots	50	18.3
BH #6	1.43	3	Deep grey clayey silt with decomposed wood.	3	16.9
		10	Brownish grey silty fine sand	6	16.5
		17	Hard, yellowish grey, clayey silt with high percentage of sand mixture	10	17.4
		25	Deep grey silty clay	16	17.7
		34	Brownish grey, silty clay with organic matters and decomposed wood	34	18
		40	Brownish grey, silty clay with steel grey spots	39	18.6

$$E_p I_p \frac{d^4 y}{dz^4} + P \frac{d^2 y}{dz^2} + \eta_h d(y - y_g) = 0 \quad (2)$$

$$y_g = g_x \cos \frac{\pi[z - L_{liq}]}{2L_{liq}} \quad (3)$$

where, η_h is the modulus of subgrade reaction in kN/m³ in liquefiable soil, y_g is the permanent ground displacement profile varying with depth along the pile length and g_x is the soil displacement which is maximum at the ground

surface and varies with depth z [51, 52]. The degradation of subgrade modulus is observed for liquefiable soil with increasing displacement and is given as [18, 52, 53]:

$$\eta_h = \eta_{hn} \times s_f \quad (4)$$

where, s_f is stiffness degradation factor in a liquefiable soil and considered as 0.01 in the present study [54]. η_{hn} is the modulus of subgrade reaction for non-liquefiable soil in MN/m³ [53, 55] and expressed as:

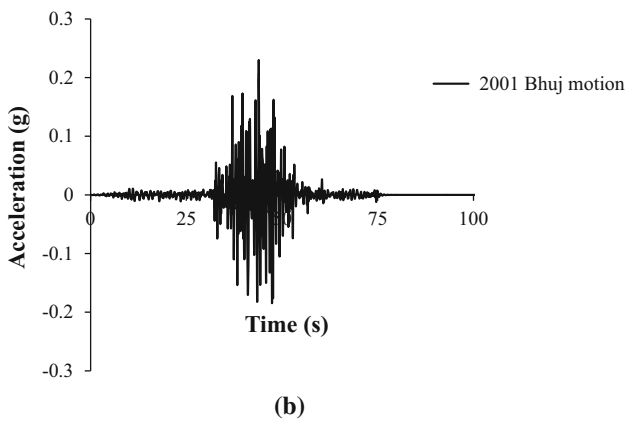
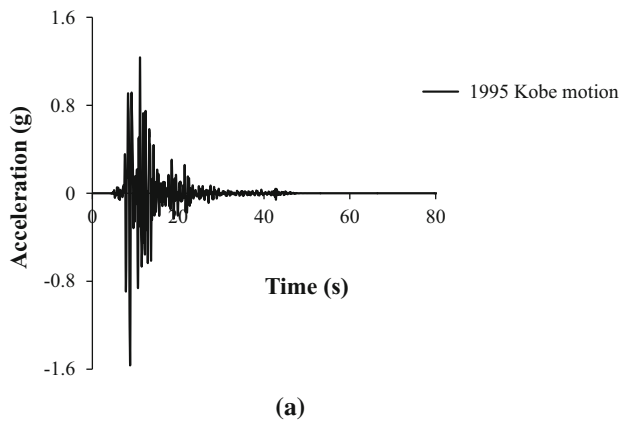


Fig. 5 Acceleration–time history at the ground surface for BH #1 when subjected to **a** 1995 Kobe and **b** 2001 Bhuj earthquake motions

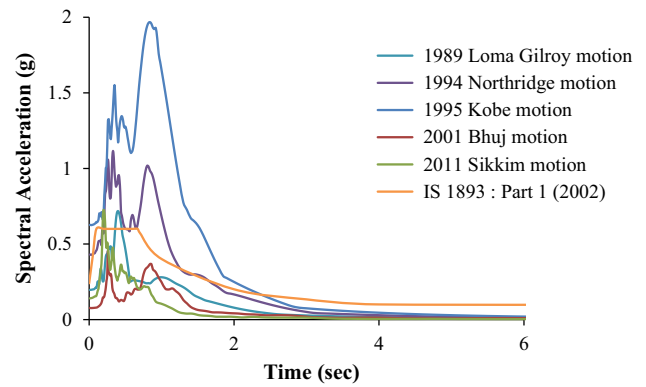


Fig. 6 Response spectrum curves at ground surface at 5% damping at BH #1 obtained in the present study

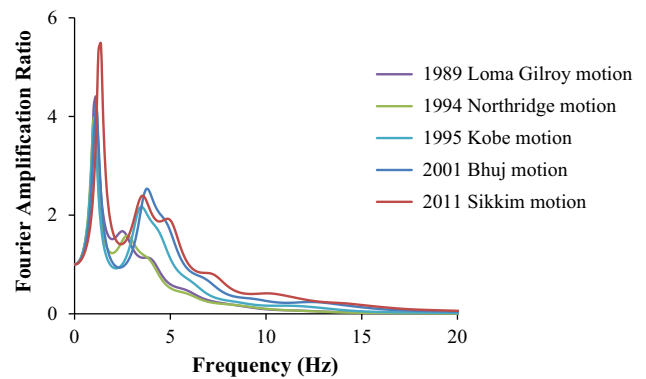


Fig. 7 Fourier amplification ratio curves at the ground surface at BH #1 obtained in the present study

Table 4 Typical dynamic soil properties calculated in the present study for BH #1

Location	GWT (m)	Depth (m)	SPT <i>N</i>	Unit weight [γ] (kN/m ³)	Layer thickness [d_i] (m)	Shear wave velocity [$V_{s,i}$] ^a (m/s)	Low strain shear modulus [G_{max}] ^b (MPa)	Average shear wave velocity [$V_{s,avg}$] [*] (m/s)	Time period of soil column [T] ^c (s)
BH #1	2.4	1.05	6	17.6	1.3	154.5	42.02	187.1	0.81
		7.4	3	16.3	6.35	118.7	22.98		
		11.9	5	17.8	4.5	144.2	37.00		
		14.15	10	18.7	2.25	187.6	65.82		
		18.4	8	19.6	4.25	172.4	58.23		
		23.6	18	20.7	5.2	234.6	113.89		
		30.4	24	19.8	6.8	261.7	135.57		
		38	35	20	7.6	302.0	182.41		

^a $V_s = 78.21 N^{0.38}$

^b $G_{max} = \frac{\gamma V_s^2}{g}$

^{*} $V_{s,avg} = \frac{38}{\sum_{i=1}^n \frac{d_i}{V_{s,i}}}$

^c $T = 4 \sum_{i=1}^n \frac{d_i}{V_{s,i}}$

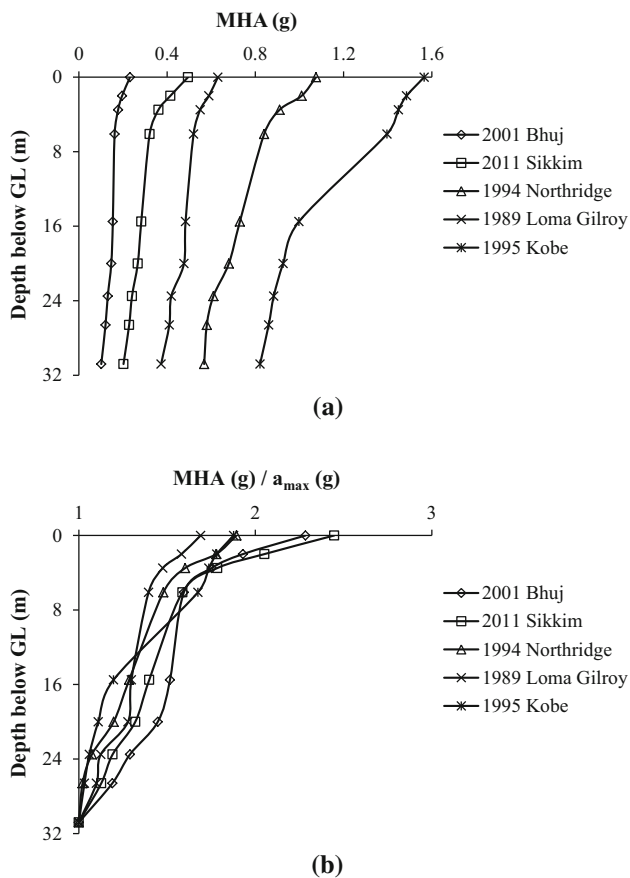


Fig. 8 Variation of **a** maximum horizontal acceleration (MHA(g)) and **b** amplification of acceleration (MHA(g)/ a_{max} (g)) along depth for BH #1

$$\eta_{hm} = 80E_o d^{-0.75} \quad (5)$$

$$E_o = 0.7N \quad (6)$$

where, E_o is the modulus of deformation in MN/m^2 , N denotes the SPT value at a particular depth and d is the diameter of the pile in cm. The governing differential equation is solved using the stiffness method based on finite element approach by considering a single pile element having two degrees of freedom (translation and rotation) at each node. A pile element is chosen and unit displacement is applied at a node keeping the rotation as zero while in the other node both displacement and rotation

are kept zero. In a similar manner by applying appropriate boundary conditions and equilibrium equations, the element stiffness matrix for each pile elements are formulated which are assembled together to obtain the global stiffness matrix, thereby relating the forces with the displacement at each pile node [36]. After calculating the displacement and rotation at various nodes, finally the force and moment of the corresponding pile elements are calculated using the load–displacement relationship. The entire procedure is accomplished by writing a code using the mathematical tool MATLAB [56] to calculate the deflection and bending moment at various nodes along the pile depth when it is subjected to the combined loadings.

A free headed single pile of M30 grade concrete with a floating tip having length 10 m, diameter 600 mm and flexural stiffness 174.3 kN m^2 is inserted into a layered soil (site BH #1), the properties of which are tabulated in Table 6. The allowable load carrying capacity of the pile is calculated to be 420 kN according to IS 2911: Part 1/Section 1 [57] and is assumed to be the safe vertical load acting on the pile top. The peak ground acceleration obtained from seismic equivalent linear ground response analysis for BH #1 site in Kolkata city are 0.63 g for 1989 Loma Gilroy motion, 1.57 g for 1995 Kobe motion, 0.24 g for 2001 Bhuj motion and 0.51 g for 2011 Sikkim motion. The lateral load (H) acting at the pile head is calculated according to Chatterjee et al. [36]. The analysis is initially conducted for inertial loading (when the pile is subjected to the input seismic motion only) and then for combined loadings. The effect of inertial loading is obtained by subtracting the deflection or bending moment observed due to kinematic loading from the corresponding deflection or bending moment observed due to combined loading [35, 58].

The influence of depth of liquefiable soil layer on pile response when subjected to combined loading is analyzed in the present study. The depth of liquefiable soil layer (L_{liq}) is varied in terms of total length of the pile (l) and 5 different combinations, i.e., $L_{liq}/l = 0.2, 0.4, 0.6, 0.8$ and 1.0, is considered. The variation of pile bending moment and pile deflection along pile depth for combined loading conditions and when subjected to 1989 Loma Gilroy and 2001 Bhuj motions for different L_{liq}/l ratio is illustrated in

Table 5 Amplification factor of acceleration at the ground surface due to different earthquake motions considered in the present study

Input earthquake motions	Peak ground acceleration [PGA] (g)	Bedrock level acceleration [a_{max}] (g)	Amplification factor (PGA/ a_{max})
1989 Loma Gilroy	0.63	0.372	1.69
1994 Northridge	1.08	0.568	1.90
1995 Kobe	1.57	0.834	1.88
2001 Bhuj	0.24	0.106	2.52
2001 Sikkim	0.51	0.202	2.28

Fig. 9 Schematic sketch of a single pile passing through liquefiable soil layer and underlain by a non-liquefiable soil layer as considered in the present study

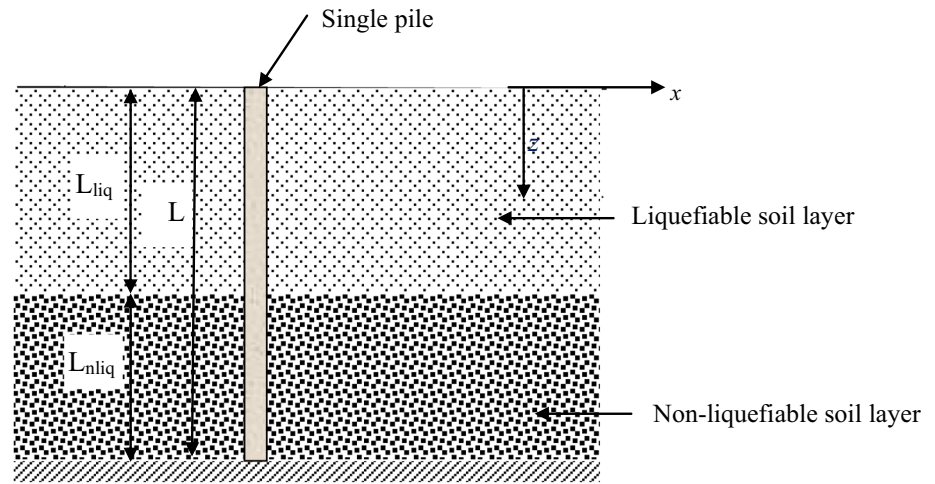


Table 6 Ground parameters at BH #1 for conducting soil-pile interaction analysis under liquefiable soil conditions

Layer no.	Soil type	SPT <i>N</i> value	<i>V_s</i> (m/s)	<i>E_o</i> (MPa)	<i>s_f</i>	η_{hm} (MN/m ³)	η_h (kN/m ³)
1	Brownish grey silty clay with organic matters	6	154.51	4.2	0.01	15.59	155.86
2	Brownish grey silty fine sand with organic matters	3	118.73	2.1	0.01	7.79	77.93
3	Deep grey silty clay with decomposed wood and organic matters	5	144.17	3.5	0.01	12.99	129.88
4	Brownish grey silty fine sand with traces of mica	10	187.61	7	0.01	25.98	259.76
5	Yellowish brown silty sand with mica	8	172.36	5.6	0.01	20.78	207.81
6	Brownish grey silty clay with kankars	18	234.57	12.6	0.01	46.76	467.57
7	Steel grey silty fine sand with mica	24	261.66	16.8	0.01	62.34	623.43
8	Brownish grey silty clay with kankars and wood	35	302.00	24.5	0.01	90.92	909.16

Fig. 10 and Fig. 11, respectively. It is observed for 1989 Loma Gilroy motion that maximum deflection is 34.6 cm when L_{liq}/l is 0.2 and it increases to 74 cm when L_{liq}/l increases to 0.6. However, when $L_{liq}/l = 1.0$, i.e., the entire soil is liquefiable the deflection at the pile head is 52.6 cm while at the pile tip is -14.8 cm. This is because the soil has lost its shear strength due to liquefaction and the pile cannot stand in such a liquefiable soil medium. In a similar manner for 2001 Bhuj earthquake motion, when the L_{liq}/l ratio increases from 0.2 to 0.6, the pile head deflection increases from 16.5 cm to 42 cm. When L_{liq}/l ratio rises to 1.0, the deflection at the pile head is 30.9 cm while at the pile tip it is -9.1 cm. The deflection is observed to be more for 1989 Loma Gilroy motion as compared to 2001 Bhuj motion due to the higher magnitude of peak ground acceleration generated from ground response analysis and greater magnitude lateral loads acting at the pile head. The maximum bending moment is observed to occur at the interface of the liquefiable and non-liquefiable soil layers and the maximum bending moment occurs when L_{liq}/l ratio is 0.6, i.e., the depth of the liquefiable soil layer is around 60% of the total pile length. Any further increase in depth

of liquefiable soil layer will cause a reduction in the magnitude of bending moment. The maximum bending moment due to 1989 Loma Gilroy motion is observed to be 702 kN m when L_{liq}/l ratio is 0.6 and it reduces to 280 kN m when L_{liq}/l ratio increases to 1.0. Similarly, for 2001 Bhuj motion, the maximum bending moment reduces from 400 to 134 kN m when L_{liq}/l ratio increase from 0.6 to 1.0. This is observed because the soil has already failed and lost its shear strength due to liquefaction before pile failure and the stresses developed in the soil under such circumstances are more than the corresponding shear strength of the soil.

The influence of 4 different seismic motions on dynamic analyses of piles having L_{liq}/l ratio 0.6 and subjected to combined loadings is analyzed in the present study and illustrated in Fig. 12. The maximum pile bending moment is observed to be 1620 kN m when subjected to 1995 Kobe motion. Similarly for 1989 Loma Gilroy, 2001 Bhuj and 2011 Sikkim earthquake motions, the bending moment generated are 702, 400 and 490 kN m respectively. The pile head deflections generated are 74, 154, 42 and 60 cm due to 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011

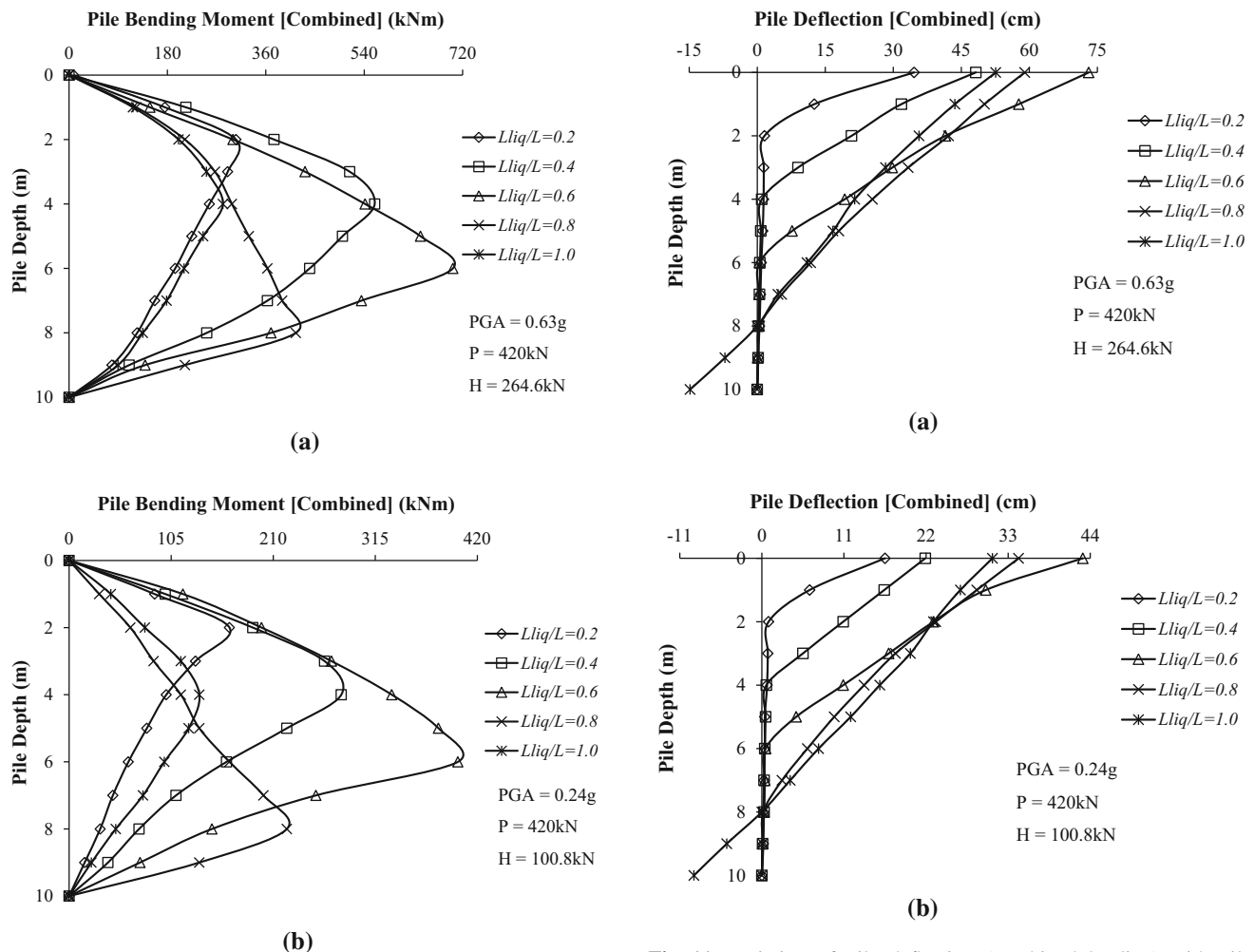


Fig. 10 Variation of pile bending moment (combined loading) with pile depth for **a** 1989 Loma Gilroy and **b** 2001 Bhuj earthquake motion for different combinations of L_{liq}/l ratio in liquefiable soil

Sikkim earthquake motions, respectively and when subjected to combined loadings. However, for kinematic loading only and when L_{liq}/l ratio is 0.6, the pile head deflection is measured to be 30.3, 50, 14 and 19.6 cm while the maximum bending moment is observed to be 248, 464, 108 and 158 kN m when subjected 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011 Sikkim earthquake motions respectively, as illustrated in Fig. 13. The influence of the inertial load is obtained after subtracting the kinematic response from the combined response [58]. The bending moment and deflections obtained due to inertial component of loading is tabulated in Table 7. The inertial component of loading of 1989 Loma Gilroy motion is observed to have 59.1 and 64.7% influence on pile head deflection and maximum pile bending moment of the total loading, while for 2001 Sikkim motions the corresponding percentage influences are 67.3 and 67.8%. The variation in pile response due to inertial loading for the various seismic

Fig. 11 Variation of pile deflection (combined loading) with pile depth for **a** 1989 Loma Gilroy and **b** 2001 Bhuj earthquake motion for different combinations of L_{liq}/l ratio in liquefiable soil

motions may be attributed to the different magnitudes of depth-wise variation of soil displacement and lateral load acting on the pile. It is also observed that although kinematic loading affects the initial pile head deflection and pile bending moment, the contribution due to inertial loading on pile response (deflection and bending moment) is significant and hence should be considered for evaluating seismic design of piles in liquefiable soil.

Conclusions

The following conclusions are obtained from the present study:

- The magnitude of shear wave velocity till 30 m depth ($V_{s,30}$) at various soil sites in Kolkata city is observed to lie in the range of 165 to 178 m/s and thereby classifying these typical Kolkata soil sites in Class E category as per NEHRP [40] guidelines. It is an

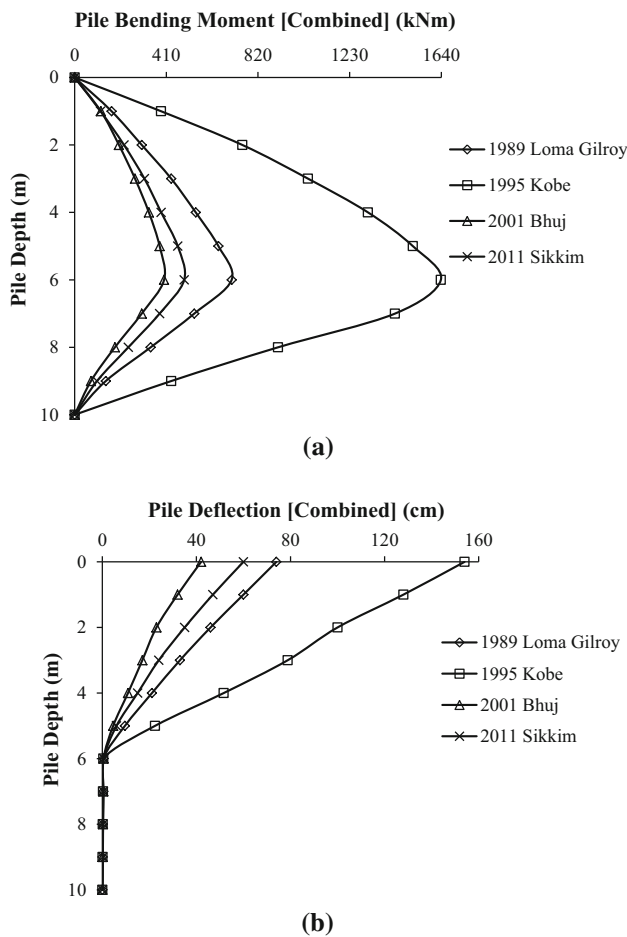


Fig. 12 Variation of **a** pile bending moment and **b** pile deflection with pile depth due to combined loading when L_{liq}/l ratio = 0.6

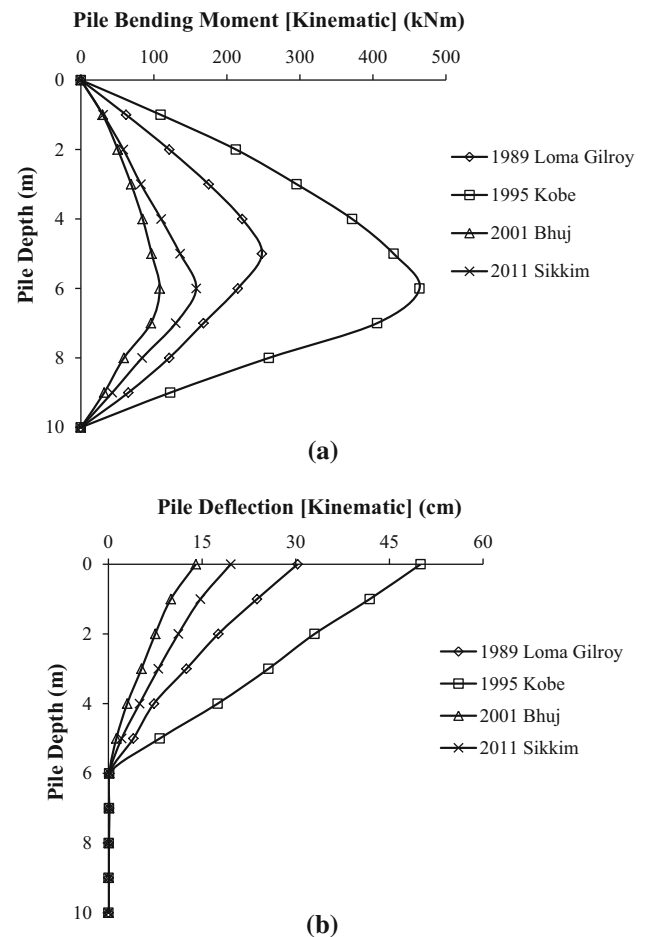


Fig. 13 Variation of **a** pile bending moment and **b** pile deflection with pile depth due to kinematic loading when L_{liq}/l ratio = 0.6

indication of the seismic risk associated with these areas.

- The seismic ground response analysis is significantly affected by the bracketed duration and frequency content of the input ground motions and the bedrock level acceleration (a_{max}) has little influence over the same. Thus 2001 Bhuj and 2011 Sikkim earthquake motions caused higher ground amplifications at various soil sites of Kolkata city as compared to 1995 Kobe motion due to the higher duration and frequency content of the former two motions.
- The amplification factor of bedrock level acceleration at the ground surface lies within 1.6–2.5 at various soil sites of Kolkata city. This is due to the soft alluvial and clayey soil layers located in Kolkata city.
- The response spectrum curves obtained in the present study for the different input earthquake motions provides valuable information like spectral acceleration which will be beneficial for geotechnical engineers for earthquake resistant design of structures like

foundations, retaining walls, embankments at various locations of Kolkata city.

- The period of spectral acceleration of 1995 Kobe motion is observed to coincide with natural period of the soil column and hence is an indication of the destruction that might occur if an earthquake of such high magnitude occurs in Kolkata city in future.
- The magnitude of pile bending moment and pile head deflection is considerably increased in liquefiable soil layers due to the degradation in stiffness of the soil which reduces the shear strength of the soil significantly.
- The depth of liquefiable soil layer has a profound impact on pile head deflection and pile bending moment. Maximum bending moment is observed to occur at the interface of the liquefiable and non-liquefiable soil layers. Further, bending moment is maximum when depth of the liquefiable soil layer is around 60% of the total pile length, i.e., L_{liq}/l ratio is 0.6.

Table 7 Magnitudes of pile head deflection and pile bending moment obtained for different component of loadings in the present study

Parameters	Input seismic motion	Combined loading	Kinematic loading	Inertial loading
Pile head deflection ($L_{liq}/l = 0.6$) (cm)	1989 Loma Gilroy (PGA = 0.63 g)	74	30.3 40.9%	43.7 59.1%
	1995 Kobe (PGA = 1.57 g)	154	50 32.5%	104 67.5%
	2001 Bhuj (PGA = 0.24 g)	42	14 33.3%	28 66.7%
	2011 Sikkim (PGA = 0.51 g)	60	19.6 32.7%	40.4 67.3%
Maximum pile bending moment ($L_{liq}/l = 0.6$) (kN m)	1989 Loma Gilroy (PGA = 0.63 g)	702	248 35.3%	454 64.7%
	1995 Kobe (PGA = 1.57 g)	1620	464 28.6%	1156 71.4%
	2001 Bhuj (PGA = 0.24 g)	400	108 27%	292 73%
	2011 Sikkim (PGA = 0.51 g)	490	158 32.2%	332 67.8%

- In addition to kinematic loadings, the influence of inertial loads should be considered for seismic analysis and design of piles passing through liquefiable soil.

Thus the present study is important for geotechnical engineers since it considers the influence of local soil sites for seismic analysis of pile foundations in Kolkata city and proper care should be taken for design of piles passing through liquefiable soil layers.

Acknowledgements The authors are grateful to C. E. Testing Company Pvt. Limited, Kolkata for providing authentic soil data of Kolkata city and its surrounding region to carry out this research work. The authors also thank the two anonymous reviewers for their valuable and constructive suggestions to improve the original manuscript.

References

1. Naik N, Choudhury D (2015) Deterministic seismic hazard analysis considering different seismicity levels for the state of Goa, India. *Nat Hazards* 75(1):557–580
2. IS 1893-Part 1 (2002) Criteria for earthquake resistant design of structure. Bureau of Indian Standards, New Delhi, India
3. Anbazhagan P, Thingbaijam KKS, Nath SK, Kumar JNN, Sitharam TG (2010) Multi-criteria seismic hazard evaluation for Bangalore city, India. *J Asian Earth Sci* 38:186–198
4. Boominathan A, Dodagoudar GR, Suganthi A, Maheswari RU (2008) Seismic hazard assessment of Chennai city considering local site effects. *J Earth Syst Sci* 117(S2):853–863
5. Hanumanthrao C, Ramana GV (2008) Dynamics soil properties for microzonation of Delhi, India. *J Earth Syst Sci* 117(S2):719–730
6. Choudhury D, Shukla S (2011) Probability of occurrence and study of earthquake recurrence models for Gujarat state in India. *Disaster Adv* 4(2):47–59
7. Shukla J, Choudhury D (2012) Seismic hazard and site-specific ground motion for typical ports of Gujarat. *Nat Hazards* 60(2):541–565
8. Shukla J, Choudhury D (2012) Estimation of seismic ground motions using deterministic approach for major cities of Gujarat. *Nat Hazards Earth Syst Sci* 12:2019–2037
9. Raghukanth STG, Sreelatha S, Dash SK (2008) Ground motion estimation at Guwahati city for an M_w 8.1 earthquake in the Shillong Plateau. *Tectonophysics* 448:98–114
10. Govindaraju L, Bhattacharya S (2012) Site-specific earthquake response study for hazard assessment in Kolkata city, India. *Nat Hazards* 61:943–965
11. Chatterjee K, Choudhury D (2016) Influences of local soil conditions for ground response in Kolkata city during earthquakes. *Proc Natl Acad Sci India Sect A Phys Sci*. doi: [10.1007/s40010-016-0265-1](https://doi.org/10.1007/s40010-016-0265-1)
12. Jishnu RB, Naik SP, Patra NR, Malik JN (2013) Ground response analysis of Kanpur soil along the Indo-Gangetic Plains. *Soil Dyn Earthq Eng* 51:47–57
13. Phanikanth VS, Choudhury D, Reddy GR (2011) Equivalent-linear seismic ground response analysis of some typical sites in Mumbai. *Geotech Geol Eng* 29(6):1109–1126
14. Desai SS, Choudhury D (2014) Spatial variation of probabilistic seismic hazard for Mumbai and surrounding region. *Nat Hazards* 71(3):1873–1898
15. Desai SS, Choudhury D (2015) Site-specific seismic ground response study for nuclear power plants and ports in Mumbai. *Nat Hazards Rev ASCE* 16(4):04015002-1–04015002-13
16. Hamada M (1992) Large ground deformations and their effects on lifelines: 1964 Nigata earthquake. Technical report NCEER 92-001, Chapter 3 of Hamada M and O'Rourke TD, National Center for Earthquake Engineering Research, pp 1–123
17. Ishihara K (1997) Geotechnical aspects of the 1995 Kobe earthquake: Terzaghi Oration. In: Proceedings of the 14th international conference on soil mechanics and foundation engineering, Hamburg, Germany
18. Tokimatsu K, Oh-oka H, Satake K, Shamoto Y, Asaka Y (1998) Effects of lateral ground movements on failure patterns of piles in the 1995 Hyogoken-Nambu earthquake. In: Proceedings of geotechnical earthquake engineering and soil dynamics III, Reston, pp 1175–1186
19. Finn WDL, Thavaraj T, Fujita N (2001) Piles in liquefiable soils: seismic analysis and design issues. In: Proceedings of 10th international conference on soil dynamics and earthquake engineering, vol 4, pp 169–178
20. Bhattacharya S (2003) Pile instability during earthquake liquefaction. PhD Thesis, University of Cambridge, Cambridge, UK
21. Bhattacharya S, Madabhushi SPG, Bolton MD (2004) An alternative mechanism of pile failure in liquefiable deposits during earthquakes. *Geotechnique* 54(3):203–213
22. Dash SR, Bhattacharya S, Blakeborough A (2010) Bending-buckling interaction as a failure mechanism of piles in liquefiable soils. *Soil Dyn Earthq Eng* 30(1–2):32–39

23. Knappett JA, Madabhushi SPG (2012) Effects of axial load and slope arrangement on pile group response in laterally spreading soils. *J Geotech Geoenviron Eng ASCE* 138:799–809
24. Abdoun T, Dobry R, O'Rourke TD, Goh SH (2003) Pile foundation response to lateral spreads: centrifuge modelling. *J Geotech Geoenviron Eng ASCE* 129(10):869–878
25. Brandenberg SJ, Boulanger RW, Kutter BL, Chang D (2005) Behavior of pile foundations in laterally spreading ground during centrifuge tests. *J Geotech Geoenviron Eng ASCE* 131:1378–1391
26. Motamed R, Towhata I, Honda T, Tabata K, Abe A (2013) Pile group response to liquefaction induced lateral spreading: E-Defense large shake table test. *Soil Dyn Earthq Eng* 51:35–46
27. Dobry R, O'Rourke MJ (1983) Discussion on seismic response of end-bearing piles by R Flores-Berrones and RV Whitman. *J Geotech Eng Div ASCE* 109(5):778–781
28. Mylonakis G (2001) Simplified model for seismic pile bending at soil layer interfaces. *Soils Found* 41(4):47–58
29. Nikolaou S, Mylonakis G, Gazetas G, Tazoh T (2001) Kinematic pile bending during earthquakes: analysis and field measurements. *Geotechnique* 51(5):425–440
30. Liyanapathirana DS, Poulos HG (2005) Seismic lateral response of piles in liquefying soil. *J Geotech Geoenviron Eng ASCE* 131:1466–1479
31. Liyanapathirana DS, Poulos HG (2005) Pseudo-static approach for seismic analysis of piles in liquefying soil. *J Geotech Geoenviron Eng ASCE* 131:1480–1487
32. Maiorano RMS, de Sanctis L, Aversa S, Mandolini A (2009) Kinematic response analysis of piled foundation under seismic excitation. *Can Geotech J* 46:571–584
33. Haldar S, Babu GLS (2010) Failure mechanisms of pile foundations in liquefiable soil: parametric study. *Int J Geomech ASCE* 10:74–84
34. Maheshwari BK, Sarkar R (2011) Seismic behavior of soil-pile-structure interaction in liquefiable soils: parametric study. *Int J Geomech ASCE* 11(4):335–347
35. Phanikanth VS, Choudhury D, Reddy GR (2013) Behaviour of single pile in liquefied deposits during earthquakes. *Int J Geomech ASCE* 13(4):454–462
36. Chatterjee K, Choudhury D, Poulos HG (2015) Seismic analysis of laterally loaded pile under influence of vertical loading using finite element method. *Comput Geotech* 67:172–186
37. Chatterjee K, Choudhury D, Rao VD, Mukherjee SP (2015) Dynamic analyses and field observations on piles in Kolkata city. *Geomech Eng Int J* 8(3):415–440
38. Chatterjee K, Choudhury D (2015) Analytical and numerical approaches to compute the influence of vertical load on lateral response of single pile. *Jpn Geotech Soc Spec Publ* 2(36):1319–1322
39. Kumar A, Choudhury D (2016) DSSI analysis of pile foundations for an oil tank in Iraq. *Proc Inst Civil Eng Geotech Eng ICE Lond* 162(2):129–138
40. NEHRP (National Earthquake Hazards Reduction Program) (2003) Recommended provisions for seismic regulations for new buildings and other structures—part 1: provisions. Prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report FEMA 450), Washington DC
41. Chatterjee K, Choudhury D (2013) Variations in shear wave velocity and soil site class in Kolkata city using regression and sensitivity analysis. *Nat Hazards* 69(3):2057–2082
42. Shiuly A, Narayan JP (2012) Deterministic seismic microzonation of Kolkata. *Nat Hazards* 60:223–240
43. Sitharam TG, Anbazhagan P (2008) Site characterization using geotechnical and geophysical techniques for seismic microzonation of urban areas. In: Proceedings of international geotechnical conference on development of urban areas and geotechnical engineering, Saint Petersburg, Russia, vol I, pp 131–147
44. Maheshwari RU, Boominathan A, Dodagoudar GR (2010) Use of surface waves in statistical correlations of shear wave velocity and penetration resistance of Chennai soils. *Geotech Geol Eng* 28:119–137
45. Mhaske SY, Choudhury D (2011) Geospatial contour mapping of shear wave velocity for Mumbai city. *Nat Hazards* 59:317–327
46. Nath SK, Thingbaijam KKS, Raj A (2008) Earthquake hazard in Northeast India—a seismic microzonation approach with typical case studies from Sikkim Himalaya and Guwahati city. *J Earth Syst Sci* 117:809–831
47. Kramer SL (2005) Geotechnical earthquake engineering. Pearson Education, London
48. Ordonez GA (2004) SHAKE2000: A computer program for the 1D analysis of geotechnical earthquake engineering problems. User's manual, GeoMotions LLC Lacey, Washington USA
49. IS 1893 Part 1 (2002) Criteria for earthquake resistant design of structures: part 1 general provisions and buildings. Bureau of Indian Standards, New Delhi
50. Hetenyi M (1946) Beams on elastic foundation: theory with applications in the fields of civil and mechanical engineering. The University of Michigan Press, Ann Arbor
51. Meera RS, Shanker K, Basudhar PK (2007) Flexural response of piles under liquefied soil conditions. *Geotech Geol Eng* 25:409–422
52. Tokimatsu K (1999) Performance of pile foundations in laterally spreading soils. In: Proceedings of 2nd international conference on earthquake geotechnical engineering, Lisbon, Portugal, pp 957–964
53. Choudhury D, Phanikanth VS, Mhaske SY, Phule RR, Chatterjee K (2015) Seismic liquefaction hazard and site response for design of piles in Mumbai city. *Indian Geotech J* 45(1):62–78
54. Ishihara K, Cubrinovski M (1998) Performance of large-diameter piles subjected to lateral spreading of liquefied deposits. In: 13th Southeast Asian geotechnical conference, Taipei, Taiwan
55. JRA Part V (1996) Specification for highway bridges: seismic design. Japan Road Association
56. MATLAB (2012) Programming, version 7. The Math Works Inc
57. IS 2911 Part1 Section 1 (2010) Indian standard code of practice for design and construction of pile foundations. Bureau of Indian Standards, New Delhi
58. Tokimatsu K, Suzuki H, Sato M (2005) Effects of dynamic soil-pile structure interaction on pile stresses. *J Struct Constr Eng* 70(587):125–132