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TECHNICAL NOTE

Influence of site-specific soil amplification on seismic response of piles in liquefiable soils

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Abstract

Kolkata city in eastern India is spread along the banks of Hooghly River in a north–south direction and having typical alluvial soil which is generally soft and thick in nature. The city is situated in seismic zones III and IV and is an implication of moderate to high seismic risk. Hence in the present study, the influence of ground response analysis and subsequent soil amplification in the design of pile foundations in liquefiable soils is thoroughly discussed. One-dimensional equivalent linear ground response analysis of Kolkata city is conducted using SHAKE2000 computer program and 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions being the chosen input ground motions. The spectral acceleration at a damping ratio of 5% is seen to be 0.41 g, while the amplification factor of maximum horizontal acceleration is found to be 2.98 when 2001 Bhuj motion is the selected earthquake motion. The high magnitudes of soil amplification are attributed to the alluvial soil in maximum parts of the city. The analysis is further extended to earthquake resistant analysis of pile foundation embedded in liquefying and non-liquefying soil strata and exposed to combined loadings. The maximum bending moment is noticed at the boundary of the liquefying and non-liquefying soil layers with the depth of liquefying soil layer being almost 65% of total pile length. The importance of deflection and bending moment of the pile foundation as important parameters in seismic analysis of deep foundations is portrayed in the current study. The present results and design charts can be used by engineers for designing pile foundations against earthquake forces.

Keywords Pile · Liquefaction · Ground response analysis · Earthquake · Bending moment · Amplification

Introduction

The fracture of a fault below the surface of the earth generates body waves travelling in various directions from the source. These waves reach the interfaces between various geological materials, with contrasting mechanical and physical properties, and undergo reflection and refraction, before striking the ground surface almost vertically. According to Kramer [14], this phenomenon of computing the dynamic response of soil layers in an earthquake-prone area and estimating the response spectrum for design of various geotechnical structures in the region is called ground response analysis [1, 3, 6, 8, 10, 17, 18, 22, 26, 27]. The challenging part of ground response analysis involves determination of spectral acceleration (S_a) and maximum horizontal acceleration at ground surface (MHA) of the particular area, while the uncertainties lie in predicting the actual rupture mechanism of the faults, the seismic wave velocity and energy transmission from the source to soil site. The results of an unambiguous ground response analysis in a seismic-prone area provides a geotechnical engineer with important parameters like soil amplification, liquefaction susceptibility, fundamental frequency for evaluating the safety in design of slopes, embankments, retaining walls and foundations of structures.

Soil–pile interaction in an earthquake-prone area and subsequent pile foundation failure because of liquefaction is significantly affected by soil conditions existing at the location, induced pore water pressure, input ground motion and applied loads. Piles in liquefiable soils are subjected to both vertical loads and lateral loads, thereby increasing the susceptibility of the pile to buckling failure due to considerable degradation of stiffness. The investigation on the performance of pile foundations during recent earthquakes

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has revealed a significant role of soil-structure interaction on the behaviour of pile foundations and geotechnical structures under seismic conditions [4, 5, 7, 9, 13, 15, 19, 20, 23–25]. Most of the previous researchers have carried out seismic analysis of piles in liquefiable soils considering the presence of either vertical or lateral loads only and the input ground motion as sinusoidal in nature. The optimum depth of liquefiable soil layer having a profound influence on pile response in liquefiable soils has also not been addressed by previous researchers. Further, the analysis and behaviour of piles subjected to liquefaction and subsequent lateral spreading involves the determination of the safe load carrying capacity of the pile and deflection to ensure serviceability criteria is attained. This ensures that piles in earthquake-prone areas and subjected to combined loadings neither undergo excessive deflection nor bearing capacity failure.

According to IS 1893: Part 1 [11], Kolkata city in eastern India is situated at the boundaries of seismic Zone III and Zone IV. The city is spread along the banks of Hooghly River in a north–south direction and having typical alluvial soil which is predominantly soft and thick in origin and hence vulnerable to soil amplification when exposed to various seismic motions, applied at the bedrock level. The previous researchers have used seismic motions having bedrock level acceleration of 0.16 g to implement ground response analysis of Kolkata city. However, the influence of widely varying earthquake parameters like frequency content, duration and maximum horizontal acceleration, which were not considered by previous researchers, has been addressed in the present study. In the present study, 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions are the input ground motions. The maximum horizontal acceleration for the various input ground motions determined using seismic equivalent linear ground response analysis is further used to calculate the inertial load on the pile head along with the lateral kinematic loads, for different depths of liquefiable soil layers, which is another major contribution of the present study. Hence, a detailed methodology for the analysis of single piles in seismically active areas, considering the effect of input ground motions, local soil conditions and bending–buckling interaction owing to the presence of both vertical and lateral loads, has been carried out in the current study for an urban city like Kolkata. The results obtained include the depth-wise variation of deflection and bending moment for pile foundations subjected to combined loading conditions.

Seismic one-dimensional ground response analysis

The ground motions were applied at the bedrock level which was considered as rigid. The energy dissipation arising due to seismic waves getting reflected at the interface of the soil–bedrock was not elucidated. The soil layers were assumed to be horizontal and extending till infinity [3], and one-dimensional equivalent linear ground response analysis was carried out using SHAKE2000 [21] computer program. The modulus reduction and damping ratio curves considered in the present study for clay having different plasticity indices and sand are illustrated in Figs. 1 and 2, where modulus reduction (G/G_{max}) and damping ratio (ξ) are expressed as functions of cyclic shear strain (γ). These curves are regenerated and recalculated based on the given soil data and used in SHAKE2000 computer program.

Figure 3 illustrates the effect of local soil conditions of Kolkata city on acceleration time history generated on the ground surface. The MHA on the ground surface is 0.543 g when 1989 Loma Gilroy motion, having an acceleration of 0.372 g at the bedrock level (a_{max}), is the selected input motion. Similarly for 2011 Sikkim motion, with an acceleration of 0.202 g at the bedrock level, the resulting acceleration is 0.438 g at the ground surface. Hence, the bedrock level acceleration is amplified by 2.21 and 2.98 times, when exposed to 1995 Kobe and 2001 Bhuj earthquake motions, respectively.

The acceleration response spectrum curves at the surface of the ground for different input seismic motions at 5% damping ratio and its comparison with the response spectrum as recommended in IS 1893: Part 1 [11] are

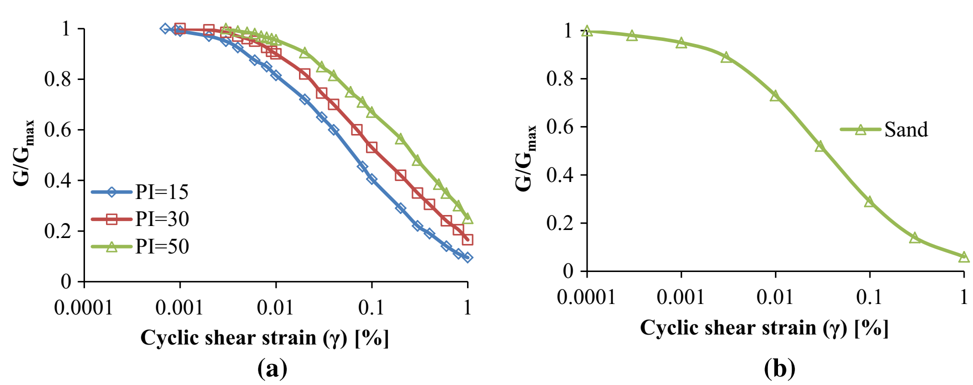


Fig. 1 Modulus reduction curves for **a** clay having different plasticity indices **b** sand

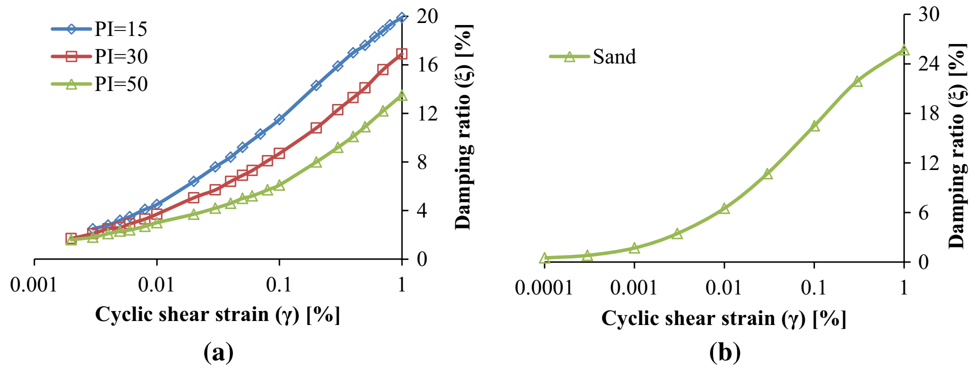


Fig. 2 Damping ratio curves for **a** clay having different plasticity indices **b** sand

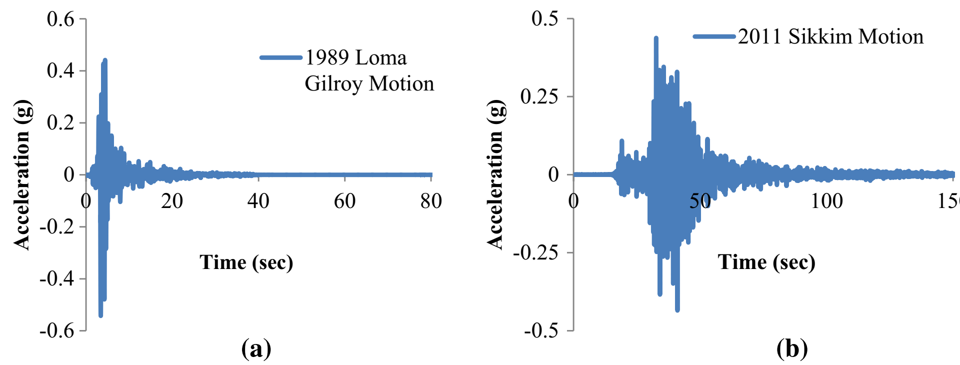


Fig. 3 Acceleration time history at the ground surface due to **a** 1989 Loma Gilroy and **b** 2011 Sikkim earthquake motions

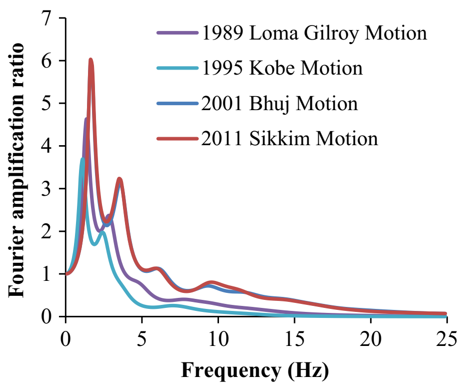


Fig. 4 Fourier amplification ratio curves at the ground surface

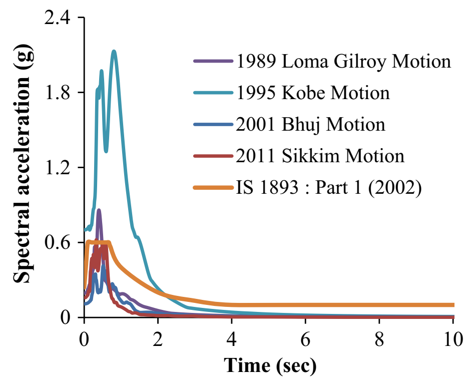


Fig. 5 Response spectrum curves at the ground surface at 5% damping ratio

illustrated in Fig. 4. The maximum spectral acceleration (S_a) is seen for 2011 Sikkim motion and is 0.62 g at a time period of 0.35 s. In a similar manner for 1989 Loma Gilroy, 1995 Kobe and 2001 Bhuj seismic motions, the equivalent magnitudes of peak spectral acceleration are 0.86 g at 0.39 s, 2.12 g at 0.75 s and 0.41 g at 0.52 s, respectively. Hence, it can be inferred that while 2011 Sikkim motion is having a profound impact on soil layers with small time period, 1995 Kobe motion is detrimental for tall structures and resting on soft soils having a longer time period.

The distribution of Fourier amplification ratio with frequency is shown in Fig. 5. The maximum amplification of 5.96 and 6.02 is observed for 2001 Bhuj and 2011 Sikkim motions, respectively, while 1995 Kobe motion produced a low amplification of 3.69. This can be attributed to the higher frequency content and bracketed duration of the first two motions as compared to 1995 Kobe motion.

The depth-wise variation of MHA below ground level at a soil site of Kolkata city and the corresponding amplification of a_{max} at various depths are illustrated in Fig. 6. 1995 Kobe motion generated MHA of 1.85 g at the ground surface, thereby indicating an amplification factor (f) of 2.21. Similarly, the MHA at the surface of ground obtained

Table 1 Amplification factor (*f*) of bedrock level acceleration of different seismic motions

Input seismic motion	a_{max} (g)	MHA (g)	*f*
1989 Loma Gilroy	0.372	0.543	1.46
1995 Kobe	0.834	1.85	2.21
2001 Bhuj	0.106	0.316	2.98
2011 Sikkim	0.202	0.438	2.17

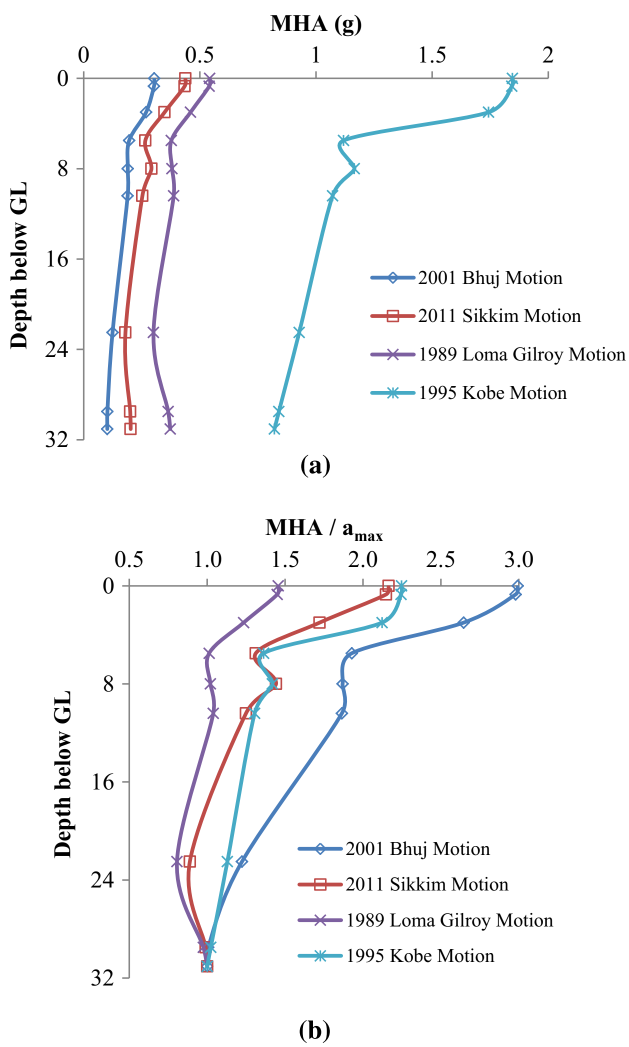


Fig. 6 Variation of **a** maximum horizontal acceleration **b** amplification factor of acceleration along depth

for 2001 Bhuj motion is 0.316 g, resulting in the amplification of bedrock level acceleration by 2.98 times. Table 1 tabulates the amplification factor (*f*) of a_{max} at the surface of ground for various seismic motions accounted for in the current study. Hence, the key influence of local soil in amplifying bedrock level acceleration is clearly emphasized in the present study. Further, due to the presence of alluvial soil across Kolkata city, high amplification factors lying between 1.5 and 3 are obtained in the present study which clearly indicates the necessity of appropriate earthquake resistant analysis of pile foundations in Kolkata city.

Seismic response of single piles in liquefiable soil

The analysis of a single pile (having diameter (d), length (l) and flexural stiffness E_pI_p) embedded in a liquefying soil layer (having thickness L_{liq}) with a non-liquefying soil layer (of thickness L_{nliq}) underlying it and subjected to a combination of vertical load (V), lateral load (H) and horizontal ground displacement (g_x) is carried out in the current study, as shown in Fig. 7. The differential equation for calculating the depth-wise (z) lateral deflection (y) and bending moment (M) in the pile for the above-mentioned loading conditions is given as:

$$E_pI_p\frac{d^4y}{dz^4} + V\frac{d^2y}{dz^2} + \eta_h dy = \eta_h dy_g \tag{1}$$

$$y_g = g_x \cos\frac{\pi\left(z - L_{\text{liq}}\right)}{2L_{\text{nliq}}} \tag{2}$$

$$\eta_h = \eta_{hn} \times s_f \tag{3}$$

$$\eta_{hn} = 80E_o d^{-0.75} \tag{4}$$

$$E_o = 0.7\text{ N} \tag{5}$$

where η_h denotes the coefficient of subgrade modulus in kN/m^3 in liquefying soil, s_f is the stiffness degradation factor in liquefying soil and taken as 0.01, η_{hn} is the coefficient of subgrade modulus in non-liquefiable soil in MN/m^3, E_o is the deformation modulus in MPa, and N denotes SPT value. The governing differential Eq. (1) is solved by adopting the finite element method, after discretizing the pile into smaller segments, each having height h, and the solution is given as:

$$\begin{aligned} y(z) &= c_1e^{\beta\frac{z}{h}}\cos\alpha\frac{z}{h} + c_2e^{-\beta\frac{z}{h}}\cos\alpha\frac{z}{h} + c_3e^{\beta\frac{z}{h}}\sin\alpha\frac{z}{h} \\ &\quad + c_4e^{-\beta\frac{z}{h}}\sin\alpha\frac{z}{h} + \frac{\eta_h dg_x}{E_pI_p\left(\frac{\pi}{2L_{\text{nliq}}}\right)^4 - V\left(\frac{\pi}{2L_{\text{nliq}}}\right)^2 + \eta_h d} \\ &\quad \times \left[\cos\frac{\pi\left(z - L_{\text{liq}}\right)}{2L_{\text{nliq}}}\right] \end{aligned} \tag{6}$$

where

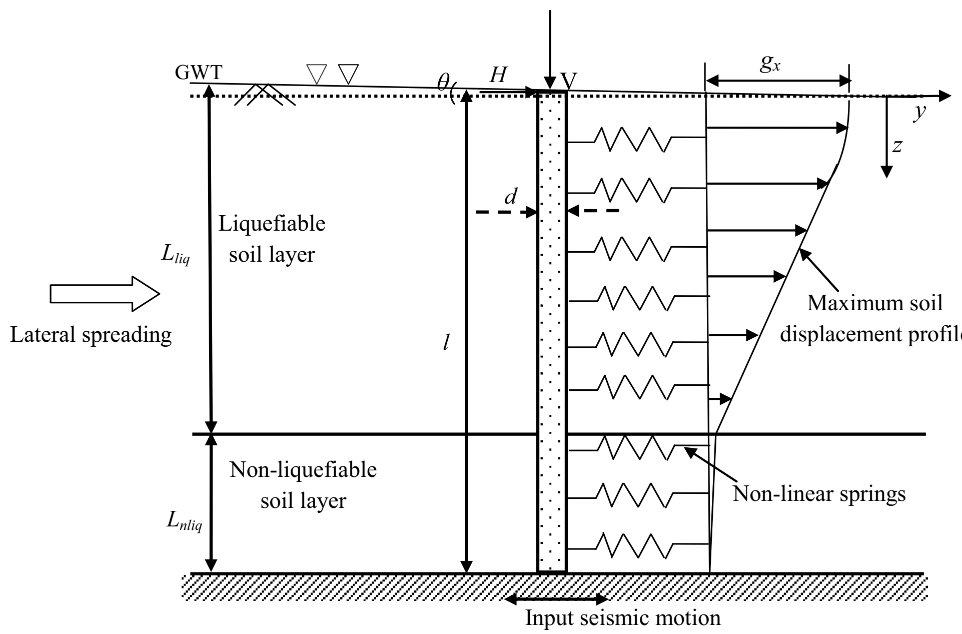
$$\alpha = h\sqrt{\sqrt{\frac{\eta_h d}{4E_pI_p}} + \frac{V}{4E_pI_p}} \tag{7a}$$

Table 2 Parameters for conducting soil–pile interaction analysis in liquefiable soil

Depth (m)	Soil type	SPT N value	V_s (m/s)[a]	E_o (MPa)	s_f	η_{hn} (MN/m^3)	η_h (kN/m^3)
0.8	Deep grey silty sand	4	132.45	2.8	0.01	10.39	103.90
2.0	Brownish grey silty fine sand	5	144.17	3.5	0.01	12.99	129.88
2.8	Steel grey silty clay with mica	6	154.51	4.2	0.01	15.59	155.86
6.7	Brownish grey silty fine sand	8	172.36	5.6	0.01	20.78	207.81
8.3	Medium light blackish silty clay	10	187.61	7	0.01	25.98	259.76
11.8	Brownish grey silty clay	14	213.20	9.8	0.01	36.37	363.67
15.0	Steel grey silty fine sand with mica	29	281.17	20.3	0.01	75.33	753.31

[a] $V_s = 78.21N^{0.38}$ [2]

Fig. 7 Schematic sketch of pile subjected to various loadings



$$\beta = h\sqrt{\sqrt{\frac{\eta_h d}{4E_p I_p}} - \frac{V}{4E_p I_p}} \tag{7b}$$

c_1, c_2, c_3 and c_4 are the unknown integrating constants and determined from the boundary conditions applied at the nodal points of each pile element, having two degrees of freedom per node, i.e., translation and rotation, as explained in [4]. After determining the integrating constants, the element stiffness matrix $[k]$ of each pile elements is calculated and assembled together to form the global stiffness matrix $[K]$ for the entire pile length. The force and bending moment at various nodes along the pile depth are obtained, and finally, the rotation and displacement at various nodes are calculated using the mathematical tool MATLAB [1, 16].

A 10 m long, 600 mm diameter, M 30 grade concrete pile, having free head with a floating tip and flexural stiffness 174.5 kNm^2, is embedded into a two-layered soil having properties as given in Table 2. According to IS 2911: Part 1 Section 1 [12], the allowable load carrying capacity of the pile (V) is determined as 425 kN. The lateral loads (H) are calculated by multiplying the allowable load carrying capacity of the pile with the MHA at the ground surface, determined from equivalent linear ground response analysis and applied at the pile top [15]. Thus, the lateral loads for different input seismic motions are calculated to be:

$$\text{1989 Loma Gilroy}: \quad H = 425 \times 0.543 = 231\ \text{kN} \tag{8a}$$

$$\text{1995 Kobe}: \quad H = 425 \times 1.85 = 786\ \text{kN} \tag{8b}$$

$$\text{2001 Bhuj}: \quad H = 425 \times 0.316 = 134\ \text{kN} \tag{8c}$$

$$\text{2011 Sikkim}: \quad H = 425 \times 0.438 = 186\ \text{kN} \tag{8d}$$

The thickness of liquefying soil layer (L_{liq}) is expressed as the function of total pile length (l) as $L_{\text{liq}}/l = 0.25$, 0.50, 0.625. 0.75 and 1. The analysis is implemented for combined loadings (inertial and kinematic loadings acting simultaneously) in both liquefying and non-liquefying soils exposed to various input ground motions, and results are shown as variation of bending moment and pile displacement with pile length.

Influence of depth of liquefiable soil layer on dynamic response of pile

It is seen from Fig. 8 that the maximum bending moment increases from 650 to 1635 kNm as L_{liq}/l ratio rises from 0.25 to 0.625 with 1995 Kobe motion as the input motion. With further increase in L_{liq}/l ratio to 1, the maximum bending moment reduces to 760 kNm. In a similar manner, the deflection at the pile head rises from 80 cm to 169 cm and reduces to 98 cm, as L_{liq}/l ratio changes from 0.25 to 0.625 and finally to 1, as illustrated in Fig. 9. This may be attributed to the failure of soil occurring before pile failure and loss in soil strength due to liquefaction. A similar nature of depth-wise variation in bending moment and

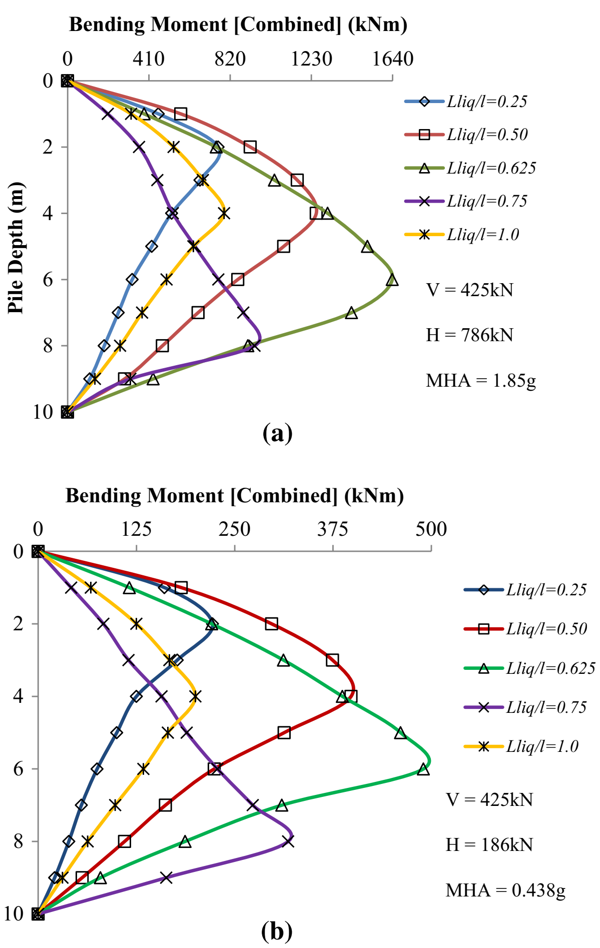


Fig. 8 Variation of bending moment with pile depth for different depths of liquefiable soil layer and when subjected to **a** 1995 Kobe and **b** 2011 Sikkim motions

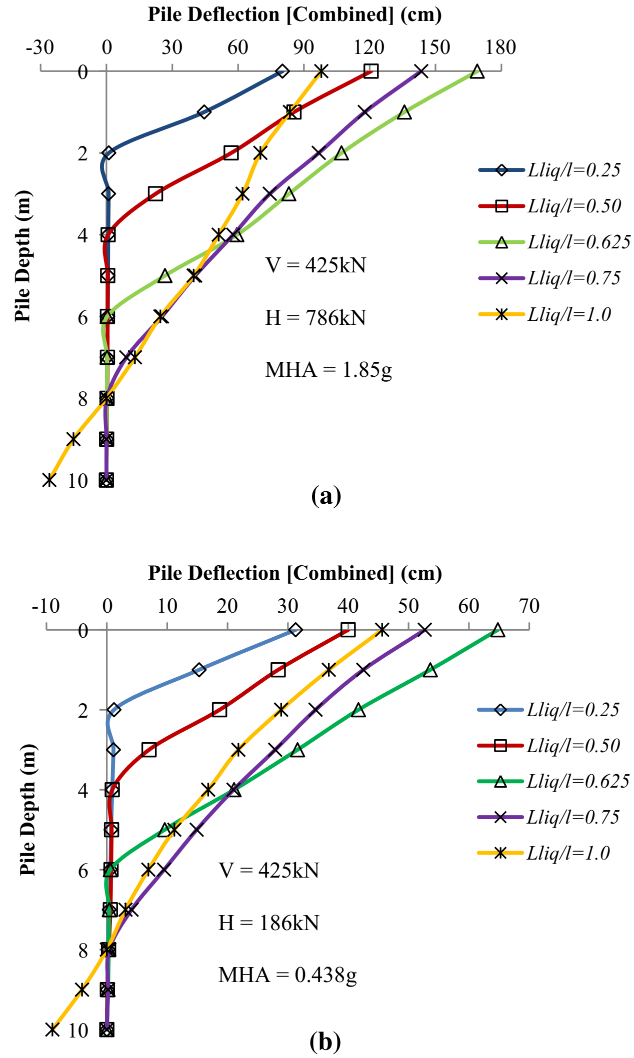


Fig. 9 Variation of deflection with pile depth for different depths of liquefiable soil layer and when subjected to **a** 1995 Kobe and **b** 2011 Sikkim motions

deflection is also noticed for 2011 Sikkim seismic motion, as portrayed in Figs. 8 and 9.

Comparison of pile behaviour in liquefiable and non-liquefiable soil

The response of the pile passing through non-liquefying and liquefying soil layer with L_{liq}/l ratio = 0.625 is evaluated, and the amplification factor (a) defined as the ratio of maximum pile head deflection or bending moment in liquefiable soil for a particular loading to that in non-liquefiable soil under similar loading conditions is given in Table 3. It is seen from Fig. 10 that the maximum bending moment due to 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions is 680 kNm, 1635 kNm, 380 kNm and 487 kNm, respectively, when both vertical and lateral loads are acting at the pile top and L_{liq}/l

Table 3 Pile response in liquefiable (L_{liq}/l ratio = 0.625) and non-liquefiable soil

Seismic motions	Liquefiable soil		Non-liquefiable soil		Amplification factor (a)	
	Deflection (cm)	Bending moment (kNm)	Deflection (cm)	Bending moment (kNm)	Deflection	Bending moment
1989 Loma Gilroy	72.1	680	7.6	410	9.5	1.7
1995 Kobe	169	1635	15.4	585	11	2.8
2001 Bhuj	40.8	380	3.1	100	13.2	3.8
2011 Sikkim	66.2	487	4.3	154	15.4	3.2

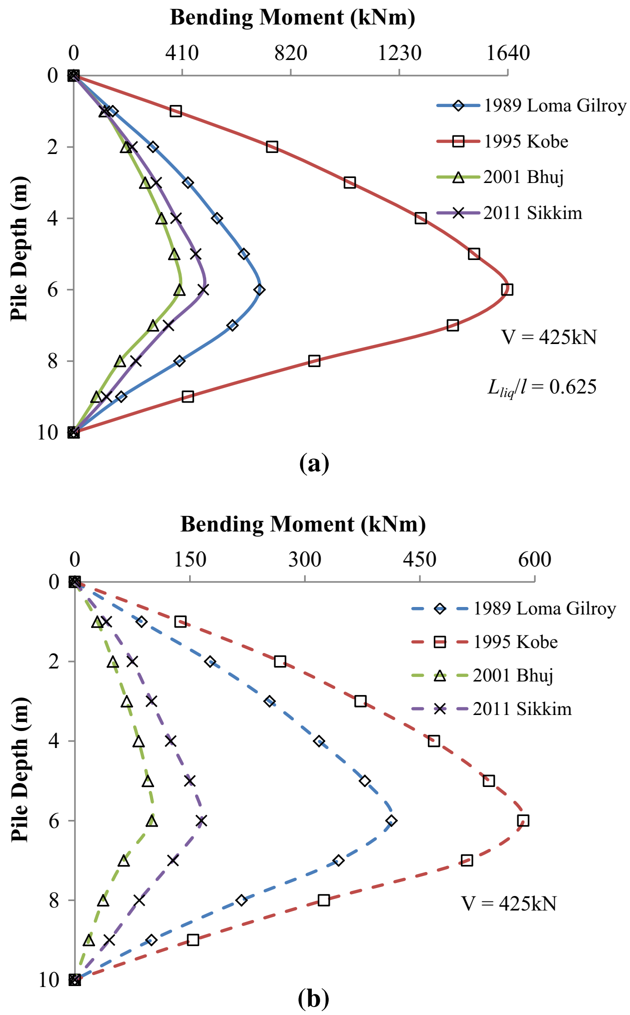


Fig. 10 Variation of pile bending moment with depth in **a** liquefiable soil for L_{liq}/l = 0.625 and **b** non-liquefiable soil

ratio = 0.625. However, in case of pile passing through non-liquefiable soil the bending moments reduce to 410 kNm, 585 kNm, 100 kNm and 154 kNm for the given sequence of motions. The amplification factors for bending moment for 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions are 1.7, 2.8, 3.8 and 3.2 and for deflection are 9.5, 11, 13.2 and 15.4, respectively.

Conclusions

The important inferences drawn from the current study are as follows:

- The acceleration at the bedrock level of the input ground motions is amplified by 1.5–3 times on the ground surface due to the presence of alluvial soil at various locations in Kolkata city. Thus, local soil sites have a considerable impact on amplifying bedrock level acceleration and modifying the ground response.
- The bending moment is maximum at the boundaries of liquefiable and non-liquefiable soil layers, with the depth of liquefying soil layer being approximately 62.5% of the total length of the pile. The deflection and bending moment at the pile head is considerably influenced by the depth of liquefying soil and increases from 80 cm to 169 cm and 650 kNm to 1635 kNm, respectively, when subjected to 1995 Kobe motion.
- The degradation in stiffness and reduction in shear strength of the liquefiable soil increases both pile deflection and bending moment.
- The amplification factors for pile bending moment are 1.7, 2.8, 3.8 and 3.2 while for deflection are 9.5, 11, 13.2 and 15.4 for 1989 Loma Gilroy, 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions, respectively. The amplification factors for pile deflection and bending moment are more for 2001 Bhuj and 2011 Sikkim motions due to their higher frequency content and duration as compared with 1995 Kobe motion.

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