Earthquake Geotechnical Engineering for Protection and Development of Environment and Constructions – Silvestri & Moraci (Eds)
ISBN 978-0-367-14328-2

Behavior of pile group in liquefiable soil deposits during earthquake

K. Chatterjee
Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, Uttarakhand, India

ABSTRACT: Seismic analysis of 2x2 pile group embedded in liquefiable soil and underlain by non-liquefiable soil is analyzed using finite difference based geotechnical program FLAC3D. The pile group is subjected to vertical and lateral loads at the pile top and earthquake motion at the pile tip. 1995 Kobe, 2001 Bhuj and 2011 Sikkim earthquake motions are the chosen input motions. The liquefiable and non-liquefiable soil layers are modeled using Byrne (1991) and Mohr-Coulomb constitutive models, respectively while pile is modeled as an elastic material. The maximum bending moment occurs at the interface of liquefiable and non-liquefiable soil layers and depth of liquefiable soil layer is approximately 65% of total pile length. The bending moment is more for the front piles as compared to the rear piles, while the pile head deflection is observed to be uniform for all piles in the group since the piles are rigidly connected to pile cap.

1 INTRODUCTION

The interaction between pile–soil–pile in a pile group subjected to earthquake loadings is considerably affected by various parameters such as soil properties existing at the site, pile properties, orientation of piles in the group, pore-water pressure induced due to seismic loading, input earthquake motions, non-linear soil behavior, presence of vertical load and constitutive laws used to model the interaction. The interaction occurs due to the connection between the piles in the group through the adjacent soil and thereby influencing the displacement and bending moment response of other piles in the group in addition to modifying the group stiffness and damping and is called the dynamic pile group effect. The dynamic analyses and behavior of pile groups subjected to liquefaction induced lateral spreading in seismically active areas involve determining the ultimate loads to arrive at the safe load carrying capacity of the pile group and estimating the deflection to ensure the serviceabilility criteria is attained. This ensures that the pile group neither fails in bearing capacity nor undergoes excessive settlement when subjected to simultaneous actions of kinematic and inertial loadings in seismic prone areas (Chatterjee and Choudhury 2018). Thus a detailed study on seismic analysis of pile group in liquefiable soil is of considerable significance.

The analysis of bending and deflection response of pile group in liquefiable soil subjected to earthquake loading have been implemented by various researchers using experimental, numerical and theoretical approaches and assuming the input motion as uniform or random in nature (Abdoun et al. 2003; Liyanapathirana and Poulos 2005; Chatterjee et al. 2015a, b; Choudhury et al. 2015; Kumar et al. 2016; Chatterjee 2018; Pan et al. 2017). The conventional approaches of analysis of pile group considered bending failure while in reality pile foundations are also susceptible to buckling failure under the action of vertical load in liquefiable soil. Moreover, the bending failure of pile foundations is influenced by bending strength of the pile while buckling failure occurs when the vertical load acting at the top of the pile exceeds its critical value (Dash et al. 2010). Further, the damping model considered for dynamic analysis of pile group in liquefiable soil plays an important role in governing the response of the pile group (Chatterjee and Choudhury 2017). However as seen from literature, studies determining the deflection and bending response of pile groups subjected to vertical and lateral loads under dynamic conditions are scarce in literature and the present study adopts a numerical approach to bridge this gap.

2 NUMERICAL MODELING OF PILE GROUP IN LIQUEFIABLE SOIL

A 2x2 pile group comprising of 4 circular concrete piles, each having length (*l*) of 8m and diameter (*d*) 600mm is inserted into a two layered soil profile, as illustrated in Figure 1. The soil profile comprises of top liquefiable soil layer having depth 5m and underlain by a non-liquefiable soil layer of 7m thickness. The properties of the soil layers and pile are tabulated in Tables 1 and 2, respectively. The concrete piles are rigidly connected to a square concrete pile cap having dimensions 4m x 4m and thickness 0.5m, placed on the ground surface. The top layer of the ground surface is kept inclined at 5° with the horizontal to initiate lateral spreading due to liquefaction. Hence the length and width of the soil block in both directions are kept 8m, while the height of the soil block at the upslope and downslope ends are 12.7m and 12m, respectively and facilitating free flow of liquefiable soil during earthquake. The loose liquefiable soil layer and non-liquefiable soil layer is modeled using Byrne (1991) and Mohr-Coulomb constitutive models, respectively. The piles in the group are represented by an elastic model with each pile segment being modeled as two nodded structural element having six degrees of freedom per node, i.e., three translational and three rotational components along the x, y and z axes. Soil-pile interaction have been modeled in the present study using a normal response at the pile tip and shear response along the pile shaft. The shear response is modeled by placing an interface between the pile and the adjacent soil through which it is passing while the modeling of the normal response is implemented by placing an interface between the pile tip and underlying soil. As per IS 2911 Part1: Section 4 (1984), the interface friction angle (δ) is assumed to be equivalent to the soil friction angle (ϕ) while the normal stiffness (k_n) and shear stiffness (k_s) at the soil-pile interfaces are calculated as per Timoshenko and Goodier (2002) and tabulated in Table 3. The pile group-soil model is subjected to 1995 Kobe, 2001 Bhuj and 2011 Sikkim earthquake motions applied at the tip of the pile group embedded in non-liquefiable soil and vertical load (V) being applied on the pile cap, the

Table 1. Soil properties considered in the present study for dynamic analysis as per Abdoun et al. (2003)

Properties	Types of Soil	
	Liquefiable soil layer	Non-liquefiable soil layer
Young's modulus [E_s] (MPa)	18	75
Poisson's ratio [μ]	0.31	0.4
Density [ρ] (kg/m^3)	1962	2038
Relative density [D_r] (%)	40	-
Cohesion [c_u] (kPa)	-	5.1
Friction angle [ϕ] (°)	30	34.5

Table 2. Pile properties considered in the present study (IS 456 2000)

Pile Properties	Values
Grade of Concrete	M25
Young's Modulus [E_p] (MPa)	25000
Poisson's Ratio [μ]	0.2
Density [ρ] (kg/m^3)	2500
Moment of Inertia [I_p] (m^4)	0.0064

Table 3. Soil–pile interface properties considered in the present dynamic analysis as per IS 2911 Part1: Section 4 (1984) and Timoshenko and Goodier (2002)

Interface Properties	Types of Soil	
	Liquefiable soil layer	Non-liquefiable soil layer
Shear stiffness [k_s] (MN/m)	0.92	3.6
Normal stiffness [k_n] (MN/m)	12.88	50.22
Interface friction angle [δ] (°)	30	-

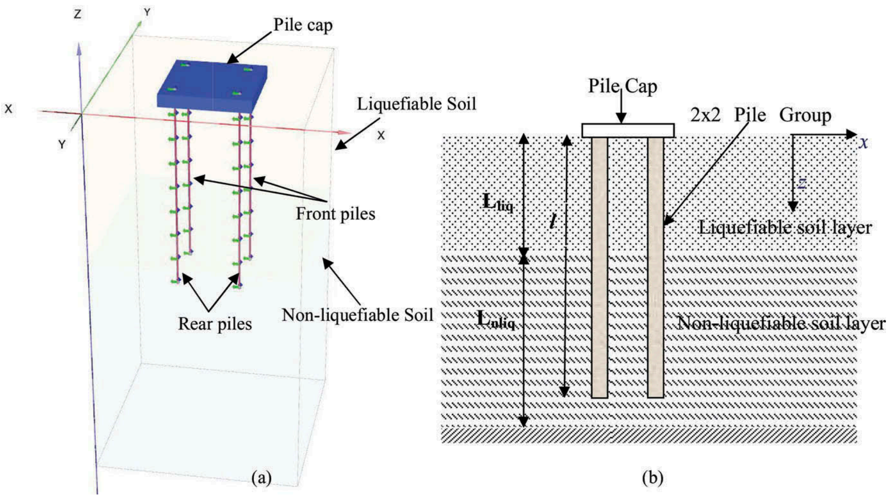

Figure 1. Pile group-soil model considered in the study in (a) three-dimensions and (b) two-dimensions

magnitude of which is 500kN. Dynamic analysis is conducted by providing free field boundary conditions and ensuring that seismic waves at the boundaries of the soil model are not reflected, with Hardin model of hysteresis damping being the input damping model.

3 RESULTS AND DISCUSSIONS

The dynamic soil structure interaction analysis is implemented using finite difference based computer program FLAC3D (2009). The analysis is carried out in two phases, i.e., kinematic phase where-in the pile group-soil model is subjected to the input ground motions only and combined loading phase where-in addition to the input ground motion, the pile group-soil model is also subjected to vertical inertial load, thereby simulating bending-buckling criteria. The depth of liquefiable soil layer (L_{liq}) in the present study is varied in terms of total pile length (l) as L_{liq}/l = 0.25, 0.50, 0.625, 0.75 and 1.0 while the spacing (s) between the piles in the group is varied in terms of diameter of the piles as s/d = 2, 3 and 4. The results obtained in the present study are illustrated in the form of variation of pile bending moment for both front and rear piles and lateral pile group displacement along pile depth for different depths of liquefiable soil layer (L_{liq}), input seismic motions and spacing between the piles in the group for both kinematic and combined loadings.

3.1 *Influence of depth of liquefiable soil layer*

It is seen from Figure 2 that for the pile group with s/d=2, L_{liq}/l = 0.25 and subjected to 2001 Bhuj motion, the maximum bending moment generated in the front pile is 12.34kNm while

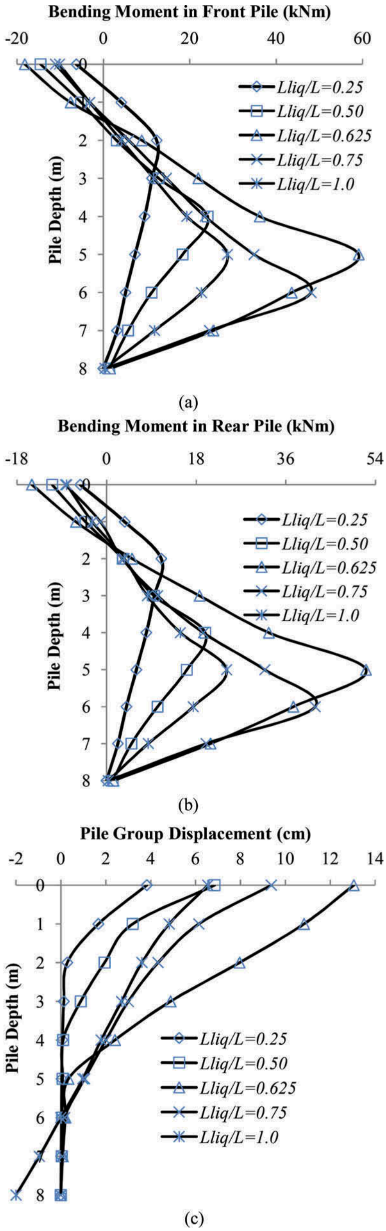

Figure 2. Pile response due to kinematic loading for different combination of L_{liq}/l ratios with s/d=2 and 2001 Bhuj as the input motion

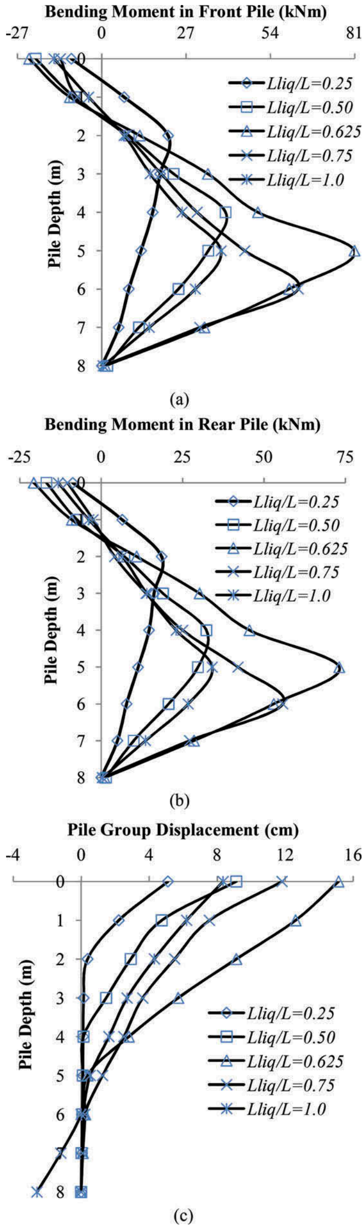

Figure 3. Pile response due to combined loading for different combination of L_{liq}/l ratios with s/d=2, V=500kN and 2001 Bhuj as the input motion

that in the rear pile is 11kNm, under kinematic loading conditions. However when L_{liq}/l ratio increases to 0.625, the maximum bending moment in the front and rear piles are 59.11kNm and 52.11kNm which reduces to 28.73kNm and 24.11kNm, respectively when the soil is completely liquefiable, i.e., *Lliq/l* ratio increases to 1. This may be attributed to soil failure occurring before pile failure when the entire soil mass in undergoing liquefaction. The displacement at the pile head changes from 3.83cm to 13.1cm to 6.55cm as *Lliq/l* ratio increases from 0.25

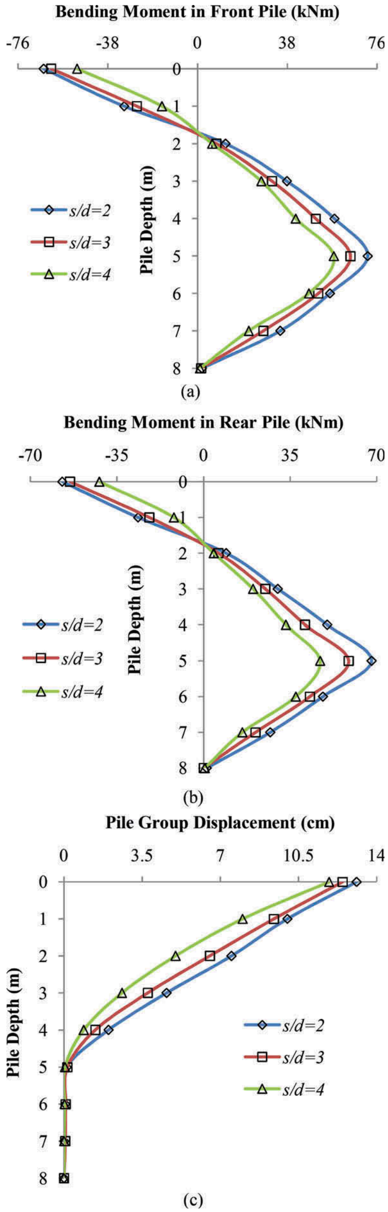

Figure 4. Pile response due to kinematic loading for different s/d ratios with L_{liq}/l = 0.625 and 2011 Sikkim as the input motion

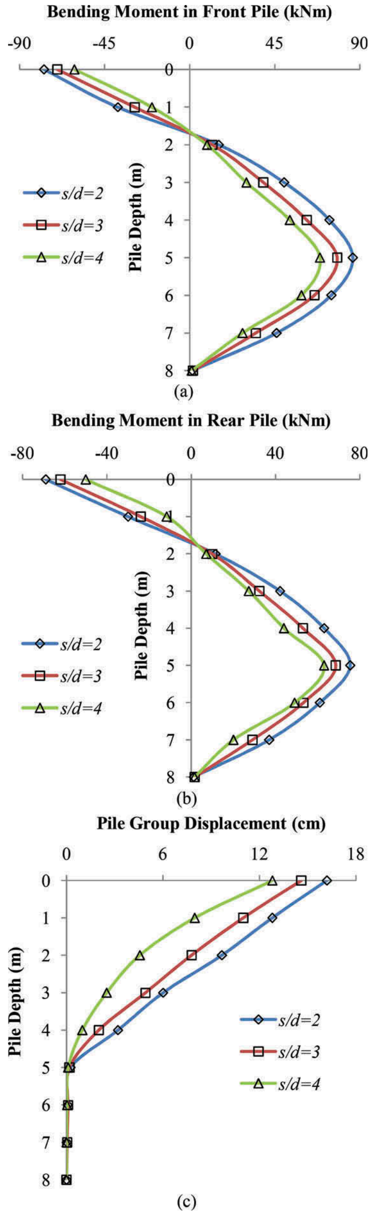

Figure 5. Pile response due to combined loading for different s/d ratios with L_{liq}/l = 0.625, V=500kN and 2011 Sikkim as the input motion

to 0.625 to 1, under kinematic loading conditions. Similarly under combined loading conditions, when in addition to an input ground motion a vertical load of 500kN is applied at the pile head, the maximum bending moment in the front pile changes from 21.28kNm to 80.91kNm to 38.21kNm, in the rear pile from 18.55kNm to 73.12kNm to 34.1kNm and the pile head displacement from 5.11cm to 15.13cm to 8.35cm, as *Lliq*/*l* ratio increases from 0.25 to 0.625 to 1, as illustrated in Figure 3. In all conditions the maximum bending moment is observed to occur at the interface of the liquefiable and non-liquefiable soil layers due to a variation in shear stiffness across the interface.

3.2 *Influence of spacing between piles in the group*

The influence of *s/d* ratios = 2, 3 and 4 on pile group response under both kinematic and combined loadings is evaluated in the present study. It is seen for *Lliq/l* = 0.625 and 2001 Sikkim motion the maximum bending moment in the front piles are 72.1kNm, 64.8kNm and 57.7kNm while that in the rear piles are 67.9kNm, 58.7kNm and 47.1kNm respectively for *s/d* = 2, 3 and 4 under kinematic conditions. The pile head displacement changes from 13.1cm to 12.5cm to 11.9cm for the same variation in *s/d* ratios, as illustrated in Figure 4. Similarly when vertical load is included in the analysis, the magnitude of bending moment in the front pile is 86.3kNm, 78kNm and 69.1kNm, in the rear pile is 75.4kNm, 68.6kNm and 63kNm while the pile head deflection is 16.2cm, 14.6cm and 12.8cm when *s/d* ratio changes from 2 to 3 to 4, as shown in Figure 5. This is because at near spacing, the piles in the group are within the displacement field of a neighboring pile which results in higher magnitudes of bending moment and pile group displacement. As the spacing increases, the distance between the adjacent piles are increased and the influence of the displacement field is significantly reduced. Moreover under dynamic loading the pile group response is also influenced by the pile and soil characteristics along with the predominant period of response spectrum of the input ground motion. As the predominant period of the input ground motion coincides with the fundamental time period of the pile group, the condition of resonance occurs and maximum displacement and bending moment are seen.

3.3 *Influence of input ground motions on pile response*

It is observed for the 2x2 pile group with *s/d* ratio 3 and L_{liq}/l ratio 0.625, the maximum bending moment in front pile and rear pile due to 1995 Kobe motion is 72.3kNm and 63.1kNm, respectively. However, for 2001 Bhuj and 2011 Sikkim earthquake motions, the bending moments developed are 56.2kNm and 64.8kNm for the front pile while for the rear pile the magnitudes are 48.6kNm and 58.7kNm, respectively, as illustrated in Figure 6. Similarly the displacement at the pile head are 14cm, 11.6cm and 12.5cm for 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions. The bending moment due to 1995 Kobe motion is higher and as compared to 2001 Sikkim motion, since the former has a higher bedrock level acceleration (a_{max} = 0.834g). Moreover when inertial loads are also included in the analysis the bending moment in the front pile increases by 18%, 29%, 21%, in the rear pile by 16.5%, 35%, 14% and pile group displacement by 7%, 18% and 17% for 1995 Kobe, 2001 Bhuj and 2011 Sikkim motions, as shown in Figure 7. Thus combined loading has a significant contribution on the lateral pile group displacement and bending moment due to *P-Δ* (load-displacement) effects.

4. CONCLUSIONS

The major conclusions drawn from the present study are as follows:

1. The deflection and bending moment in front and rear piles are influenced by thickness of liquefiable soil layers, spacing between the piles in group, input seismic motions and vertical load acting on the pile cap.
2. The maximum bending moment occurred at the interface of liquefiable and non-liquefiable soil layer and when depth of liquefiable soil layer is 62.5% of total length of pile, i.e., *Lliq/l* = 0.625. The bending moment is more for the front piles as compared to the rear piles. The pile head displacement is same since the piles are rigidly connected to the pile cap.
3. The magnitudes of bending moment in front pile and rear pile as well as lateral pile group displacement are higher in liquefiable soil due to bending-buckling criteria under the influence of combined loading as compared to kinematic interactions. This is because the

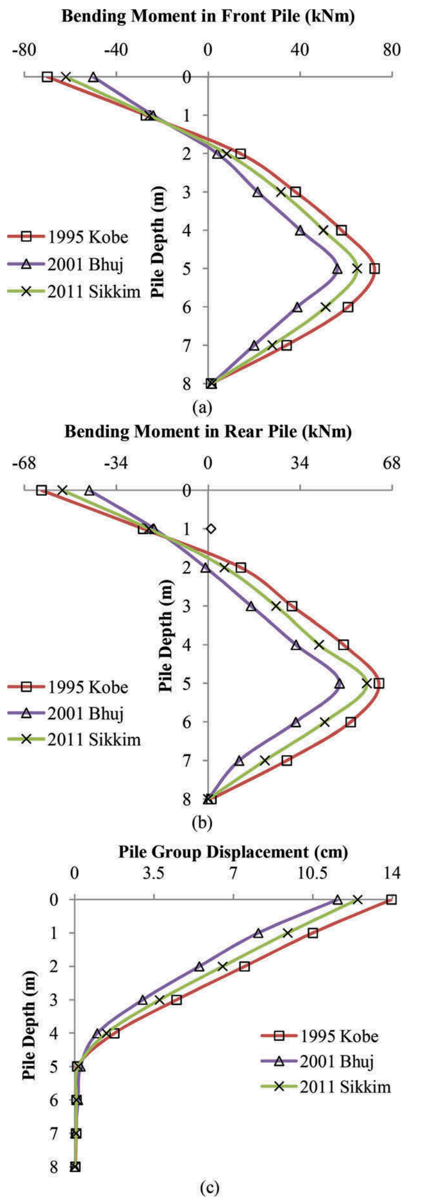

Figure 6. Pile response due to kinematic loading for different input seismic motions with $L_{liq}/l = 0.625$ and $s/d=3$

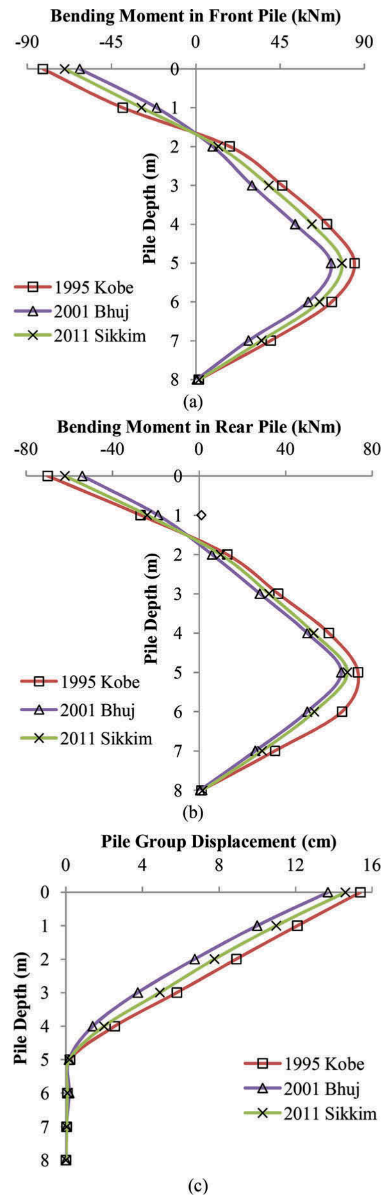

Figure 7. Pile response due to combined loading for different input seismic motions with V=500kN, $L_{liq}/l = 0.625$ and $s/d=3$

vertical load reduces the global lateral stiffness of the pile foundation due to P-Δ moment and the capacity of the pile group to resist laterally applied kinematic loading from the liquefiable soil is significantly reduced.

4. The bending moment in both front and rear piles are higher when the piles are closely spaced in the group since they are within the displacement field of the neighboring piles.

REFERENCES

Abdoun, T., Dobry, R., O'Rouke, T.D. and Goh, S.H. (2003). Pile foundation response to lateral spreads: Centrifuge modelling. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* 129 (2): 869–878.

Byrne, P. (1991). A cyclic shear-volume coupling and pore-pressure model for sand. *In Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake* Engineering *and Soil Dynamics*, St. Louis, Missouri, 47–55.

Chatterjee, K. (2018). Impact of ground response analysis on seismic behavior and design of piles in Kolkata city. *Indian Geotechnical Journal* 48(3): 459-473.

Chatterjee, K., Choudhury, D., Rao, V.D. and Mukherjee, S.P. (2015a). Dynamic analyses and field observations on piles in Kolkata city. *Geomechanics and Engineering: An International Journal* 8(3): 415–440.

Chatterjee, K., Choudhury, D. and Poulos, H.G. (2015b). Seismic analysis of laterally loaded pile under influence of vertical loading using finite element method. *Computers and Geotechnics* 67: 172–186.

Chatterjee, K. and Choudhury, D. (2017). Influence of damping models on dynamic response of pile group. *Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering*, Seoul, South Korea, 1493-1496.

Chatterjee, K. and Choudhury, D. (2018). Influence of seismic motions on behaviour of piles in liquefied soils. *International Journal for Numerical and Analytical Methods in Geomechanics* 42(3): 516-541.

Choudhury, D., Phanikanth, V.S., Mhaske, S.Y., Phule, R.R. and Chatterjee, K. (2015). Seismic liquefaction hazard and site response for design of piles in Mumbai city. *Indian Geotechnical Journal* 45(1): 62–78.

Dash, S.R., Bhattacharya, S. and Blakeborough, A. (2010). Bending-buckling interaction as a failure mechanism of piles in liquefiable soils. *Soil Dynamics and Earthquake Engineering* 30(1-2): 32–39.

FLAC3D (2009). Fast Lagrangian Analysis of Continua. Version 4.0, *Itasca Consulting Group*, Minneapolis, Minnesota, U.S.A.

IS 456 (2000). Indian standard on plain and reinforced concrete – code of practice. *Bureau of Indian Standards*, New Delhi.

IS 2911 Part1: Section 4 (1984). Indian standard code of practice for design and construction of pile foundations. *Bureau of Indian Standards*, New Delhi, India.

Kumar, A. and Choudhury, D. (2016). DSSI analysis of pile foundations for an oil tank in Iraq. *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering* 162(2): 129-138.

Liyanapathirana, D.S. and Poulos, H.G. (2005). Seismic lateral response of piles in liquefying soil. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE* 131: 1466–1479.

Pan, D., Zhang, Q., Liu, S. and Zhang, S. (2017). Analysis on response prediction of a single pile and pile groups based on the Runge-Kutta method. *KSCE Journal of Civil Engineering* 15(1): 65-75.

Timoshenko, S.P. and Goodier, J.N. (2002). Theory of elasticity. 3rd Edition, *Tata McGraw-Hill Education*.