Influence of damping models on dynamic response of pile group

Influence des modèles d'amortissement sur la réponse dynamique du groupe de pieux

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ABSTRACT: The analysis of pile groups in earthquake prone areas and subjected to seismic loading involves computing the ultimate load to arrive at the safe load and determining the maximum deflection to ensure that a serviceability criterion is reached. In the present study, a 2x2 pile group embedded in liquefiable Nevada sand layer and underlain by a non-liquefiable cemented sand layer and subjected to 2001 Bhuj earthquake motion is analyzed using finite difference based geotechnical program FLAC3D. The damping models considered in the present study includes Rayleigh damping, local damping, artificial viscosity and Hysteretic damping (Default, Hardin, sig 3 and Sig 4). Maximum pile group displacement of 9.1cm is observed when sig3 Hysteresis damping model is used while Hardin model generated dynamic bending moment of 17.5kNm and 15kNm on front and rear piles, respectively. Thus damping models have a significant influence on dynamic response of pile group founded in liquefiable soil and should be included for dynamic analysis of various geotechnical structures.

RÉSUMÉ : L'analyse des tas de groupes dans les zones sujettes aux tremblements de terre et soumis à sismique chargement implique la charge ultime pour en arriver à la charge de calcul et détermination de la déformation maximale pour s'assurer qu'un critère de facilité d'entretien est atteint. Dans la présente étude, un groupe de pieux de 2 x 2 intégré dans la couche de sable de Nevada liquéfiable et reposant sur une couche de sable cimentée non liquéfiables et soumis à 2001 Bhuj mouvement de tremblement de terre est analysé avec finis différence géotechniques selon programme FLAC3D. Les modèles amortissement considérés dans la présente étude comprend Rayleigh amortissement, local atténuant, artificiel de viscosité et de Hysteretic d'amortissement (par défaut, Hardin, sig 3 et 4 de Sig). Déplacement de groupe de pile maximale de 9,1 cm est observée lorsque sig3 hystérésis amortissement modèle est utilisé, tandis que le modèle de Hardin a généré des moment de flexion dynamique du 17.5kNm et 15kNm avant et arrière pieux, respectivement. Ainsi les modèles amortissement ont une influence significative sur la réponse dynamique de groupe de pieux fondé en sol liquéfiable et doivent être inclus pour une analyse dynamique des diverses structures géotechniques.

KEYWORDS: pile group, earthquake, liquefaction, damping, bending moment, deflection

1 INTRODUCTION

Dynamic loading like wind, wave, impact of ships against the shore and earthquake forces have a considerable effect on pile groups supporting offshore structures, transmission towers and buildings founded on soil having low bearing capacity. The analysis of pile groups in earthquake prone areas and subjected to seismic loading involves computing the ultimate load to arrive at the safe load and determining the maximum deflection to ensure that a serviceability criterion is reached.

The interaction between pile–soil–pile in a pile group under dynamic conditions is a complicated procedure and is influenced by various factors which includes soil profile and properties considered, pile properties, orientation of piles in the group, non-linear soil behavior and constitutive laws followed in the analysis, pore-water pressure induced under seismic conditions, inertial effects and kinematic interaction between pile-soil-pile. The behavior of individual piles in a pile group is strongly affected by cross interaction among the individual piles which influences the stiffness, magnitude and spatial distribution of lateral soil movements that are key parameters which control pile response in liquefiable soil (Elahi et al. 2010). The piles are connected to each other through the surrounding soil and the displacement field of one pile have a significant contribution on the displacement of the other piles existing in the group (El Sharnouby and Novak 1985), thereby giving rise to the interaction between piles in a group. This pile-soil-pile interaction redistributes the load on individual piles in the group and the pile group stiffness and damping is modified.

The analysis of single pile and pile group in liquefiable soil subjected to seismic loading using experimental, numerical and theoretical approaches have been implemented by several researchers by considering the input motion either as an uniform motion or random in nature (Abdoun et al. 2003; Liyanapathirana and Poulos 2005; Haldar and Babu 2010; Phanikanth et al. 2013; Chatterjee et al. 2015a, b; Choudhury et al. 2015; Kumar and Choudhury 2016; Kumar et al. 2016). It is also observed by researchers that bending failure of pile foundations is governed by the bending strength of the pile while buckling failure occurs when the vertical load acting at the pile top exceeds its critical value (Dash et al. 2010). Moreover 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; which has not been considered by previous researchers and has been carried out in the present study.

In the present study, numerical analysis of a 2x2 pile group embedded in liquefiable soil underlain by a non-liquefiable soil strata is carried out using FLAC3D (2009) computer program. Damping models like Rayleigh damping, local damping, artificial viscosity and Hysteretic damping (Default, Hardin, sig 3 and Sig 4) are utilized for carrying out the present analyses and variations in bending moment, ground displacement and peak ground acceleration with depth have been obtained. Thus the importance of considering proper dynamic model while executing the numerical analysis using a finite difference or finite element based geotechnical package have been illustrated in the present study.

2 NUMERICAL MODELING OF 2X2 PILE GROUP

A 2x2 pile group comprising of 4 polyetherimide piles each being 8m long (l) and 600mm diameter (d) at a spacing (s) of 3d is inserted into the centre of a 2-layered soil block. The 2-layered soil block comprised of a loose liquefiable Nevada sand layer having thickness of 5m underlain by a non-liquefiable cemented sand layer of 7m thickness. The properties of the soil layers and piles are considered as per Abdoun et al. (2003). The piles are rigidly connected to a square aluminium pile cap of dimension 3.5mx3.5mx0.5m. The top layer of the ground surface is inclined at an angle of 4.8° with respect to the horizontal for generating lateral spreading of the soil due to liquefaction. The pile-group soil model is illustrated in Figure 1.

Byrne (1991) constitutive model is used to model the loose liquefiable Nevada sand layer while Mohr-Coulomb constitutive model is selected to model the non-liquefiable slightly cemented sand layer. Further, dynamic analysis is conducted in the present study by applying free field boundary conditions and ensuring that seismic waves are not reflected at the boundaries of the soil model. The pile group-soil model is subjected to 2001 Bhuj earthquake motion, applied at the tip of the pile group embedded in cemented sand layer. Different types of damping models like Default hysteresis model, sig3 hysteresis model, sig4 hysteresis model, Hardin hysteresis model, Rayleigh damping, local damping and artificial viscosity have been utilized in the present study for carrying out dynamic analysis of pile group in liquefiable soil and the response in variation of bending moment and ground displacement is noted.

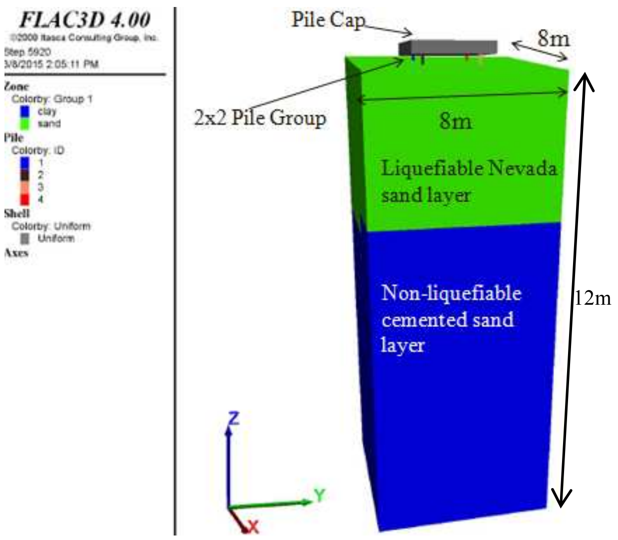

Figure 1. Pile group-soil model considered in the present dynamic analysis.

3 RESULTS AND DISCUSSIONS

The variation of bending moment with depth for both leading pile and rear pile passing through liquefiable soil for various damping models used in the present study are illustrated in Figure 2 and Figure 3, respectively. As observed, the bending moment at the pile top is -13.2kNm in front pile and -11.2kNm in the rear pile when Hardin model of Hysteresis damping is considered in the analysis. The magnitude of bending moment increases in a curvilinear manner with depth and the maximum positive bending moment is observed at a depth of 5m, ie, at the interface of the liquefiable Nevada sand layer with the non – liquefiable slightly cemented sand layer, beyond which the magnitude of bending moment decreases. The maximum positive bending moment in the front pile and rear pile are observed to be 17.5kNm and 14.9kNm, respectively, when Hardin model of Hysteresis damping is considered. However if the analysis is implemented using Rayleigh damping, the magnitude of bending moment observed in both front piles and rear piles are comparatively lower and having a magnitude of 6.38kNm and 5.4kNm, respectively. A high bending moment experienced by the piles in a pile group is critical since it is the maximum bending moment experienced by the pile when subjected to a seismic motion and leading to its failure. Hence it is important to select a proper damping model for conducting numerical analysis of pile group subjected to seismic loading.

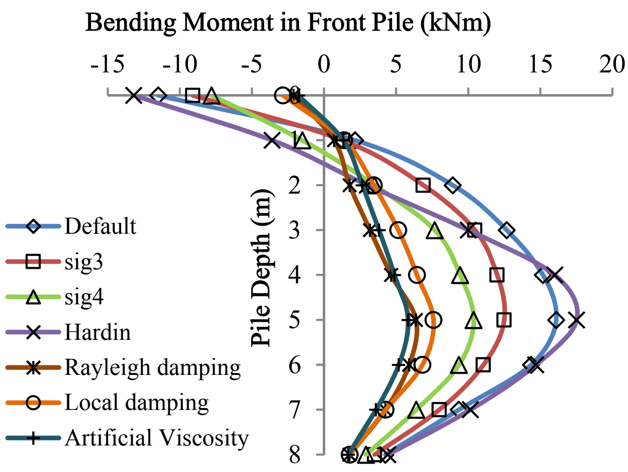

Figure 2. Variation of bending moment in leading pile with pile depth for different damping models considered in the present study and when embedded in liquefiable soil.

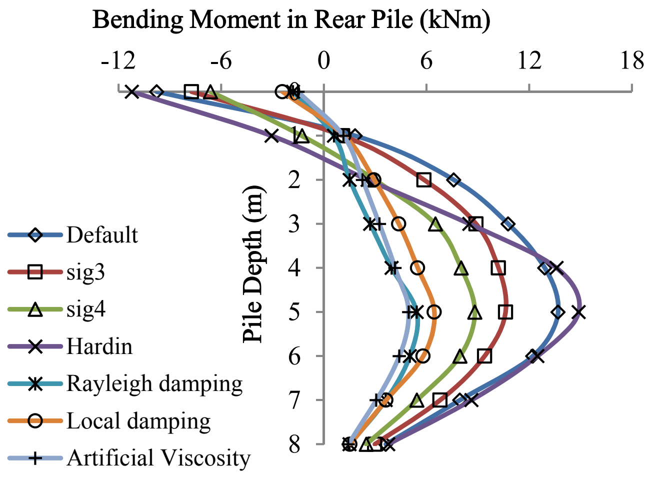

Figure 3. Variation of bending moment in rear pile with pile depth for different damping models considered in the present study and when embedded in liquefiable soil.

Figure 4 shows the variation of pile group displacement along pile depth for different damping models and when it is subjected to 2001 Bhuj motion. The pile group displacement is maximum and having a magnitude of 9.1cm when sig3 model of Hysteresis damping is considered in the analysis. The group pile displacement is maximum at the pile head, decreases along the depth of the pile and beyond a depth of 5m, the displacement is observed to be constant for a damping model considered in the analysis. This is because the non-liquefiable cemented sand layer have a higher stiffness and is denser when compared to the overlying liquefiable Nevada sand layer, which has displaced to a considerable extent when exposed to the earthquake motion.

However, the pile ground displacement due to Rayleigh damping, local damping and artificial viscosity is observed to be considerably low around 2.5cm. The variation of pile group displacement due to these three damping models are observed to be almost constant along the pile depth which implies that not much displacement is recorded when dynamic analysis is implemented using these damping models. Moreover, since the 4 piles are rigidly connected to the pile cap, all the four pile heads have undergone the same magnitude of lateral displacement.

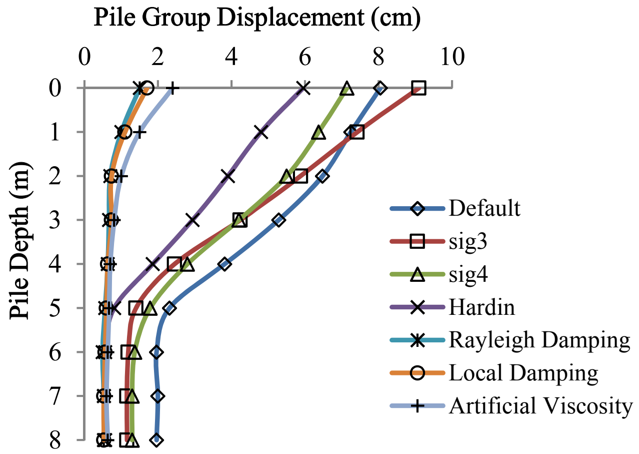

Figure 4. Variation of pile group displacement with pile depth for different damping models considered in the present study and when embedded in liquefiable soil.

The variation of maximum horizontal acceleration (*MHA*(*g*)) along depth of the pile group and subjected to 2001 Bhuj motion is shown in Figure 5. The seismic waves travel upwards through the soil layers and there is amplification of ground motion over the soft sediments due to trapping of these waves. Hence the amplitude and frequency content of the seismic waves are modified considerably while it passes through the soil deposits. The maximum horizontal acceleration at the ground surface is 2.44g when Hardin model of Hysteresis damping is considered in the analysis while Rayleigh damping generated a *MHA* of 0.70g at the ground surface. The amplification factor, which is defined as the ratio of *MHA* at the ground surface to the bedrock level acceleration of the input motion, is calculated to be 23 and 2.8 when Hardin model of Hysteresis damping and local damping is considered in the dynamic analysis of pile group, respectively.

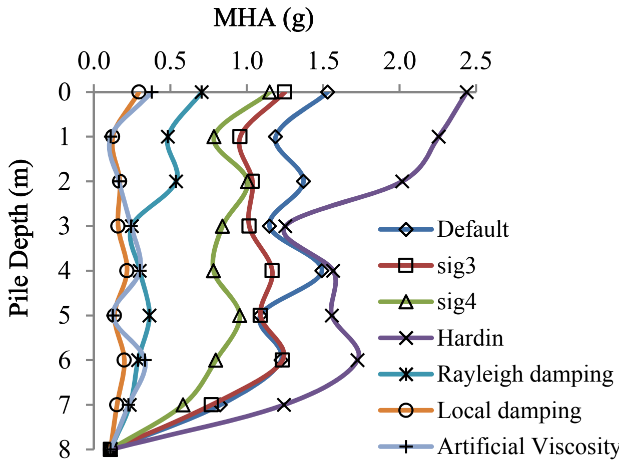

Figure 5. Variation of maximum horizontal acceleration with depth below ground level for different damping models considered in the present study and when embedded in liquefiable soil.

The variation of bending moment in both front and rear piles and when passing through non-liquefiable soil is also assessed in the present study. The maximum bending moment is observed to be 2.5kNm and 2.1kNm in front piles and rear piles respectively, when Hardin model of Hysteresis damping is used in the analysis while the magnitude becomes 0.83kNm and 0.7kNm for front and rear piles when viscous damping is considered, illustrated in Figures 6 and 7. The magnitude of bending moment in both front and rear piles are considerably reduced in non-liquefiable soil as compared to liquefiable soil due to degradation in soil stiffness and loss of shear strength of the latter, causing it flow like a fluid.

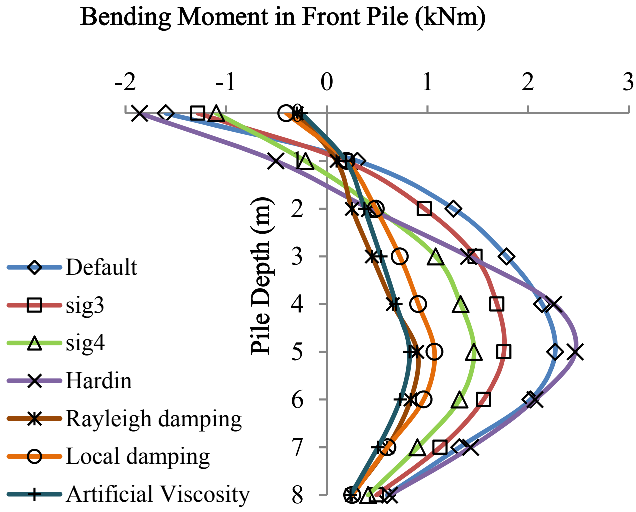

Figure 6. Variation of bending moment in leading pile with pile depth for different damping models considered in the present study and when embedded in non-liquefiable soil.

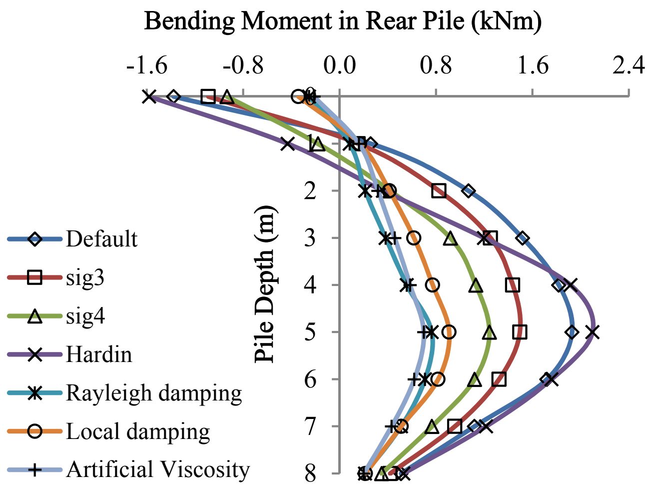

Figure 7. Variation of bending moment in rear pile with pile depth for different damping models considered in the present study and when embedded in non-liquefiable soil.

Figure 8 shows the variation of pile group displacement with pile depth in non-liquefiable soil. Maximum pile group displacement of 7.4mm is observed when sig3 model of Hysteresis damping is considered while Rayleigh damping generates the minimum pile group displacement of 1.2mm. The magnitude of lateral displacement is observed to be same for all the four pile heads since they are rigidly connected to the pile cap. In Figure 9, the variation of *MHA* with pile depth in non-liquefiable soil is illustrated. The maximum *MHA* of 0.34g is generated due to Hardin model of hysteresis damping followed by Default model generating a *MHA* of 0.21g, when considered in dynamic analysis of pile group. The maximum and minimum amplification factor is calculated to be 3.2 and 1.1 when dynamic analysis of pile group in non-liquefiable soil is implemented using Hardin model of Hysteresis damping and viscous damping, respectively. The comparison between response of pile group in liquefiable and non-liquefiable soil is tabulated in Table 1.

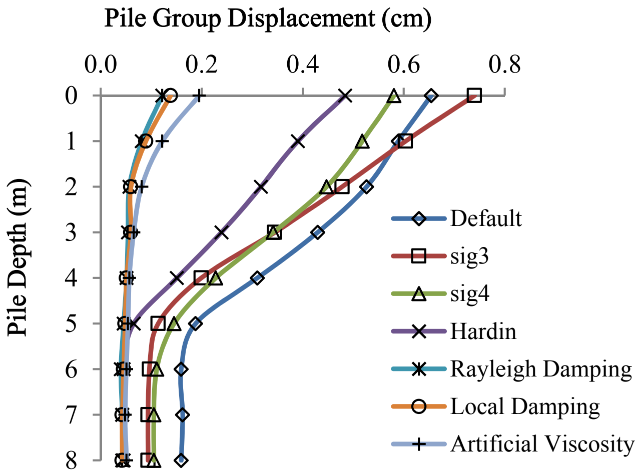

Figure 8. Variation of pile group displacement with pile depth for different damping models considered in the present study and when embedded in non-liquefiable soil.

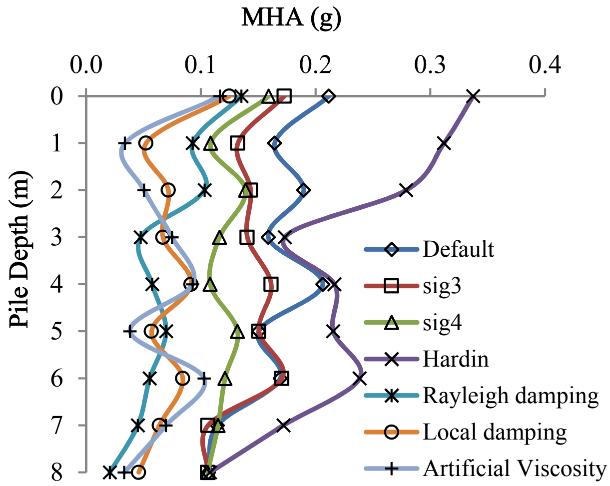

Figure 9. Variation of maximum horizontal acceleration with depth below ground level for different damping models considered in the present study and when embedded in non-liquefiable soil.

Table 1. Comparison of pile group response in liquefiable and non-liquefiable soil for different damping models considered in the present study.

Soil Type	Bending Moment (kNm)		Damping Model
	Front Pile	Rear Pile	
Liquefiable Soil	17.5	14.9	Hardin
Non-liquefiable Soil	2.5	2.1	Hardin
Soil Type	Pile Group Displacement (cm)		Damping Model
Liquefiable Soil	9.1		sig3
Non-liquefiable Soil	0.74		sig3

4 CONCLUSIONS

The major conclusions obtained from the present study are:

1. The bending moment is observed to be maximum at the interface of the liquefiable Nevada sand layer and non – liquefiable cemented sand layer for both front and rear piles.
2. The pile group displacement in liquefiable soil is observed to be considerably high when Hysteresis damping model is used in the analysis when compared to other damping models.
3. The maximum horizontal acceleration at the ground surface is amplified considerably by 23 times when Hardin model of hysteresis damping is used in the dynamic analysis and the pile group is passing through liquefiable soil.
4. Hysteresis damping is observed to give critical values of bending moment and pile group displacement when compared to Rayleigh damping, local damping and artificial viscosity in the present study. Hysteresis damping incorporates strain dependent modulus and damping functions into the FLAC3D simulation, without making any compromise on the constitutive model chosen for the dynamic analysis of pile group.
5. Hence it can be concluded from the present study that different models of Hysteresis damping gives critical results compared to other damping models and should be included during dynamic analysis of geotechnical structures. Thus different damping models have a considerable impact on the dynamic response of a pile group in both liquefiable and non-liquefiable soil.

5 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(10), 869–878.

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, Techno Press* 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, Elsevier* 67, 172–186.

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.

El Sharnouby B. and Novak M. 1985. Static and low–frequency response of pile groups. *Canadian Geotechnical Journal* 22(1), 79–94.

Elahi H., Moradi M., Poulos H.G. and Ghalandarzadeh A. 2010. Pseudostatic approach for seismic analysis of pile group. *Computers and Geotechnics, Elsevier* 37, 25–39.

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

Haldar S. and Babu G.L.S. 2010. Failure mechanisms of pile foundations in liquefiable soil: parametric study. *International Journal of Geomechanics, ASCE* 10(2), 74–84.

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, ICE* 162(2), 129–138.

Kumar A., Choudhury D. and Katzenbach R. 2016. Effect of earthquake on combined pile-raft foundation. *International Journal of Geomechanics, ASCE* 16(5), 04016013–1 to 04016013–16.

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.

Phanikanth V.S., Choudhury D. and Reddy G.R. 2013. Behaviour of single pile in liquefied deposits during earthquakes. *International Journal of Geomechanics, ASCE* 13(4), 454–462.