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Kinematic response of pile foundations in liquefiable soils

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ABSTRACT: Pile foundations often suffer excessive bending stresses induced by the vibratory deformations and the corresponding interactions of the seismic waves. Such soil-pile mutual interaction is termed as kinematic interaction and is crucial in case of liquefiable soils as additional stresses may be induced on the pile. This article aims to investigate the kinematic response of a single pile and 2×2 pile group embedded in liquefiable soil. Dynamic beam on nonlinear Winkler foundation approach (BNWF) has been adopted herein, incorporating the liquefaction behavior of soil with the hyper-elastic bilinear model. First, the BNWF model is validated against the centrifuge results. A liquefiable soil profile located in a highly active seismic region of India has been considered. Nonlinear effective stress ground response analyses were performed to account for the pore pressure generation using the experimentally derived dynamic soil properties. Seismic analysis has been performed through nonlinear time history analysis on the chosen pile configurations. An abrupt contrast in the peak bending moment has been observed at the vicinity of liquefied and nonliquefied interfaces due to the sharp stiffness contrast. The pile group showed significantly better seismic performance in terms of reduced peak displacements and bending moments compared to the single pile response, in case of liquefiable soils.

1 INTRODUCTION

Deep foundations such as piles and caissons are the most affected due to the soil structure interaction (kinematic and inertial) phenomenon and the layering effects (Nikolaou et al. 2001). Kinematic interaction is critical in case of liquefiable soils due to the possibility of lateral spreading effects (Brandenberg 2005). Understanding such complex behavior of soil-pile system is pivotal for effective seismic resistant design of pile supported structures (Bhattacharya 2003). Several advanced continuum models exist in literature in order to investigate such complex behavior, however, their applicability is often restricted due to the time and expertise requirements. Dynamic beams on nonlinear Winkler foundation approach (BNWF) seems to have reasonably captured the kinematic response and is widely used for its simplicity and less expertise required (Boulanger et al. 1999; Finn & Fujita 2002). The Dynamic BNWF analysis is basically a decoupled approach whereby the soil-pile interaction is modelled through a series of closely spaced nonlinear Winkler springs (often termed as *p-y* springs) and the analysis is performed by applying the seismic free field soil displacements. Critical aspect in the approach is the establishment of Winkler spring properties. Several studies suggested various empirical forms for the development of Winkler springs, both for nonliquefiable (API 2007; Matlock & Reese 1961) and liquefiable soils (Brandenberg 2005; Dash et al. 2017; Lombardi et al. 2016).

Based on this, objective of the present study is focused to investigate the kinematic bending response of a single pile and a 2×2 pile group embedded in liquefiable soil stratum underlain by a thick and stiff nonliquefied deposit. First, validation is performed for the BNWF model through comparison with the centrifuge results of Boulanger et al. (1999). Nonlinear time history analysis using BNWF approach was performed for the chosen configurations. The required displacement time histories are obtained from nonlinear effective stress ground response analysis. The results are presented in terms of response of the pile (displacements and bending moments) and critical observations are drawn based on the results.

2 VALIDATION

The methodology of Dynamic BNWF approach has been validated against the centrifuge test results of dynamic behavior of single pile in liquefiable soil reported by Boulanger et al. (1999). Figure 1 (a) shows the developed dynamic BNWF model. Similar soil and pile properties as reported by Boulanger (1999) were adopted. Equivalent linear site response analysis was performed using DEEPSOIL program while Boulanger et al (1999) utilized SHAKE 1991. Scaled and baseline corrected earthquake motion of Kobe 1995 event with a Peak Ground Acceleration (PGA) of 0.05g was used for the analysis. Boulanger et al. (1999) also compared the centrifuge results with the BNWF approach to understand the efficiency of Winkler springs (referred as *p-y* springs hereafter) in simulating the Seismic-Soil-Pile-Structure response. Nonlinear Winkler spring properties were obtained following the work of Matlock & Reese (1961) for the Bay Mud (surficial 6m), and API (2007) recommendations for sand layer (base 11m). Elastic radiation damping (far field springs) properties are considered from the study of Wang et al. (1998). Boulanger et al (1999) utilized a finite element program - GeoFEAP for the nonlinear time history analysis, while SAP 2000 (Computers and Structures Inc. 2015) has been used in the present validation scheme. Pile is modelled as linear elastic steel material and the soil as two joint link (spring) element, one end connected to the pile joint and the latter connected to the free field spring (Figure 1a). Gapping of the soil-pile interaction is neglected in the study, as Boulanger et al. (1999) concluded that gapping absence would only result in ±5% differences. The *p-Δ* effects were considered in the analysis. Nonlinear time history analysis was performed by inputting the obtained free field displacement histories (from site response analysis) at different depths of the pile (at the far end of the elastic *p-y* springs). Multi-support excitation option available in SAP2000 has been used for the analysis via Hilbert-Hughes-Taylor direct time history approach.

Figures 1 (b and c) presents the peak displacement and peak bending moment responses of the pile. It can be observed that the current BNWF model could reasonably simulate the response of pile both in terms of peak displacement and bending moments in comparison with the BNWF adopted by Boulanger et al. (1999). The narrow mismatch of BNWF results against the centrifuge data were attributed to the uncertainties involved in site response analysis such as neglecting the nonlinearity of the soil, pore pressures and multi- dimensionality of the seismic waves.

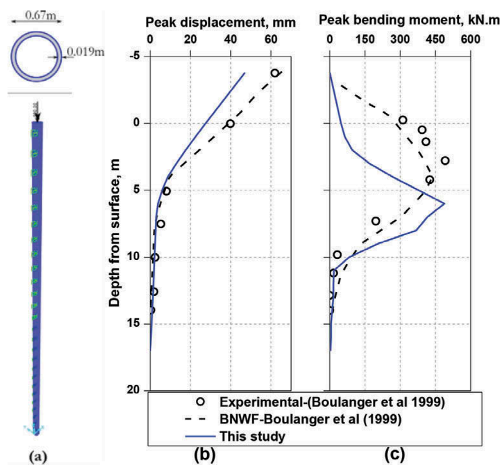

Figure 1. (a) Validated pile configuration and results of validation model in terms of (b) peak pile displacements and (c) bending moments of the pile along the depth.

3 PROBLEM STATEMENT

Present study considers two pile foundation configurations (single pile and 2×2 pile group) as shown in Figure 2a. The single pile is of 0.6 m in diameter with 26 m in length, out of which 0.5 m is above the ground level. The pile is made of concrete material with 25 MPa compressive strength. All the four piles in the 2×2 pile group are also similar in geometry and material properties. The pile cap is of 0.9 m thick with center to center spacing of the piles maintained as 1.6 m. Both the single pile and group are considered to be embedded in a three layered stratum-surficial 5 m is of loose sandy stratum and 5 to 10 m is of stiff red soil (stiff clay), followed by a thick dense sand for 20 m. Soil stratigraphy in terms SPT N values and shear strength characteristics is shown in Figure 2b and Table 1. The considered foundation configuration (single pile and 2×2 pile group) is used to investigate the kinematic behavior in different seismic conditions. The dynamic BNWF approach basically consists of three main steps: free field ground response analysis, development of *p-y* spring properties and nonlinear time history analysis using free field site response analysis results.

3.1 *Nonlinear effective stress Ground Response Analysis (GRA)*

Nonlinear effective stress GRA has been performed for the chosen soil profile using DEEPSOIL program incorporating pore water pressure generation (Hashash et al. 2016). The analysis is performed in the time domain with pressure dependent modified Kondner Zelasko (MKZ) model for the nonlinear stress-strain behavior of soil. Any nonlinear effective stress GRA studies require the soil properties such as V_s (or low strain shear modulus-G_{max}) along with the normalized shear modulus reduction ratio (G/G_{max}) and damping ratio (D) variation with shear strain, in addition to the liquefaction parameters. The shear wave velocity (V_s) of the profile is estimated by correlating it with SPT N values (Imai & Tonouchi (1982)). In case

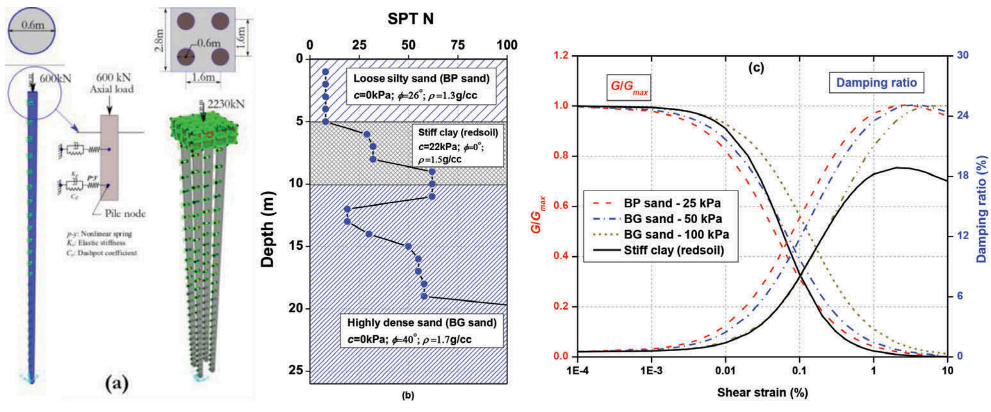

Figure 2. (a) Single pile and 2×2 pile group configuration (b) Soil profile details and (c) G/G_{max} and damping curves established.

Table 1. Design soil profile considered

Soil type	Depth (m)	Unit weight, kN/m^3	G/G_{max} and D curves adopted (σ'_m)
Loose sand	0-5	10.40	BP sand (25 kPa)
Stiff clay	5-10	14.81	Red soil
Dense sand	10-19	16.77	BG sand (50 kPa)
Dense sand	19-26	18.21	BG sand (100 kPa)

of G/G_{max} and damping ratio curves, the site-specific data is considered from the experimental work of Dammala et al. (2017) and Kumar et al. (2018). Ideally, each layer should have unique G/G_{max} and damping curves, however, as the layers are divided based on the maximum frequency travel criteria, thickness was maintained as 1m. Therefore, based on the suggestion of Zhang et al. (2005), effective confining pressure (σ'_m) dependent curves are used for sand at different depths and for clay- unique curves at a particular plasticity index is used (Table 1). Figure 2c presents the considered G/G_{max} and damping curves. Liquefaction parameters for the sand were considered from the cyclic triaxial test results conducted on the samples collected from the site (Dammala et al. 2019). In case of red soil, formulations proposed by Carlton (2014) were employed to determine the liquefaction parameters. The input motion required for the site response analysis was considered from the study of Raghu Kanth et al. (2008), whereby the motions were developed for Guwahati city for the largest recorded event in the region (1897 Shillong with a moment magnitude of 8.1). The motion was baseline corrected and linearly scaled to different peak bed rock accelerations (0.05g, 0.138g, 0.24g). Figure 3 presents the corrected and filtered motion with a peak bed rock intensity of 0.138g. All the considered ground motions were applied at the base near the bedrock and the response is monitored.

Figure 4 (a-c) presents the response of soil deposit in terms of peak ground acceleration (PGA), peak displacement and pore water pressure ratio (PWP) for the three ground motions chosen. It can be inferred that the low intensity bedrock motions (0.05g and 0.138g) showed high amplification on reaching the surface (100 to 130%) while the high intensity motion (0.24g) yielded low amplification (0.275g from 0.24g). Such phenomenon of low amplification

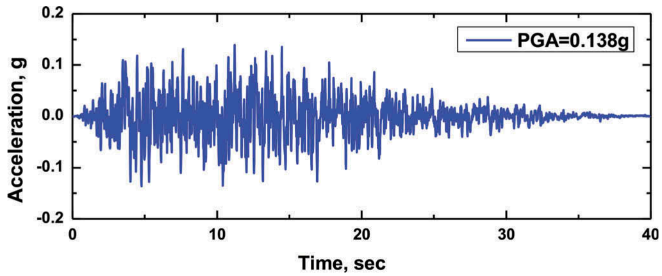

Figure 3. Considered input ground motion for the analysis (Raghu Kanth et al. (2008)).

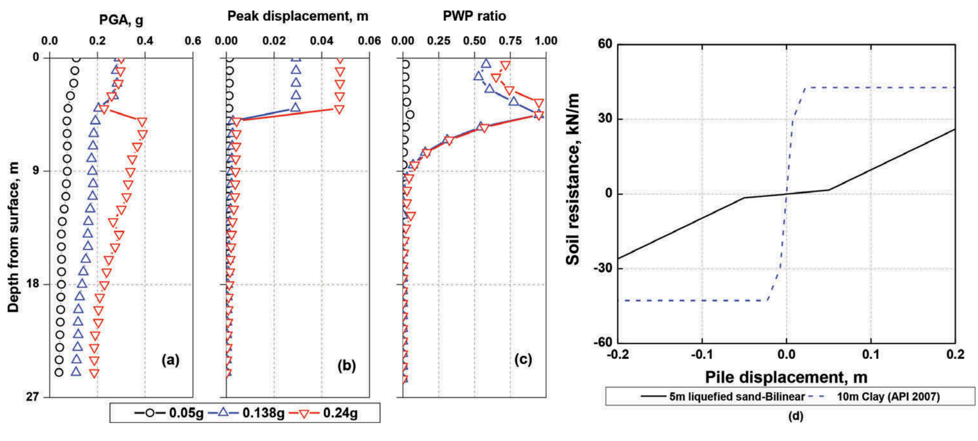

Figure 4. (a) Free field GRA results in terms of peak ground acceleration (b) peak displacement and (c) pore water pressure variation and (d) Nonlinear p-y properties for liquefiable sand at 5m depth and non-liquefiable clay at depth.

or attenuation for high intensity motions was explained by Kumar et al. (2015) as the high damping (Figure 2 c) at high induced strains (Figure 4 b). The PWP ratio (r_u>1.0) suggests complete liquefaction of the surficial loose sand layer (5 m) for both the 0.138g and 0.24g events. It is interesting to note the significant increase of peak displacement at surface for high intensity motion (0.06m for 0.24g compared to 0.03m for 0.138g) despite both events showing the same depth of liquefaction.

3.2 *Dynamic analysis of pile foundations*

The next step in the dynamic analysis of pile foundations is the generation of *p-y* curves at different depths based on soil properties. Present study employed API (2007) simplified correlations for developing the *p-y* curves in case of nonliquefiable soils (for 0.05g ground motion). However, in case of liquefiable events (0.138g and 0.24g showed liquefaction of top 5 m), the mechanics based bilinear hyperelastic model proposed by (Lombardi et al. 2014) has been adopted. Figure 4 d presents a comparison of the nonliquefiable (clay at 10 m depth) and liquefiable *p-y* springs (for sand at 5 m depth). It can be observed that the liquefiable soil doesn't hold any initial effective stress and with increased shearing, strain hardening behavior was observed due to the dilating behavior of the sandy soils (Wang et al. 1998). The commercially available structural program (SAP 2000) has been adopted for the nonlinear time history analysis. Pile is modelled as a frame element with linear elastic concrete material of M25 strength. Soil is modelled using the two jointed link element for both the elastic far field (Wang et al. 1998) and nonlinear *p-y* elements -one attached to the pile joint while the other joints to the elastic element. The other end of elastic element is fixed for all degrees of freedom and given the input as displacement histories obtained from free field GRA (see Figure 2a for further details). The base of the pile is considered hinged and the pile head is free for all degrees of freedom. Similarly, piles in the 2×2 pile group are also modelled and the pile cap of thickness 0.9 m is modelled using thick shell element with translation possibility and rotation fixity (general field case conditions). Due to the pile-soil-pile interaction possibility in the pile group or shadowing effects, Brown et al. (1988) and Rollins et al. (2005) suggested a reduced *p-y* curves for the piles in the group with a multiplication factor of 0.5. The same is considered here for modelling the *p-y* curves of piles in the group.

Analysis is performed through multi-support excitation via Hilbert-Hughes-Taylor direct time history analysis. The axial load acting on the pile is determined as the safe load corresponding to the ultimate axial capacity estimated using the soil properties. The p-Δ effects are considered in through the geometric nonlinearity and the frequency dependent global damping is considered for the analysis. Gapping behavior is ignored for the analysis as suggested by Boulanger et al. (1999).

4 RESULTS AND DISCUSSION

4.1 *Single pile response*

The displacement response of single pile for the events 0.05g and 0.24g are presented in Figures 5 a and b respectively. It must be noted that the event 0.05g did not show any liquefaction for the site and therefore, the plastic deformations did not exist in the soil and also in the pile at any depth (Figure 5 a). However, as the surficial 5 m loose sandy stratum liquefied for the subsequent events (0.138g and 0.24g), permanent displacements are induced in the soil leading to high induced stresses in the pile. Such high stresses induced plastic deformation of the pile till the depth of liquefaction (Figure 5 b). Similar permanent pile displacements in liquefiable soils were observed by Wilson et al. (2000) in centrifuge tests and in continuum finite difference model (Haldar & Babu (2010)). Figure 5 (c and d) presents the peak displacement and peak bending moment response of the pile for different seismic events. It is clear that the peak displacements and peak bending moments of the pile increased with the intensity of the motion, due to the increased free field displacements with increased intensity of motion

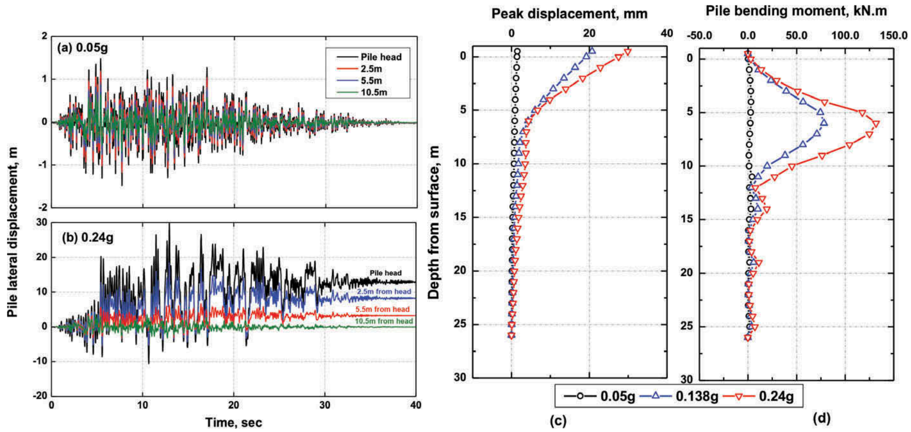

Figure 5. (a) Displacement histories of single pile for 0.05g and (b) 0.24g events and variation of (c) peak displacement and (d) peak bending moment.

(Figure 4 b). The peak pile displacements count to 50mm for 0.24g event while the pile deflected 21mm in case of 0.138g event. In case of peak bending moments, maximum moments are observed at the vicinity of the liquefiable and nonliquefiable layer, showcasing the effect of sharp stiffness contrast as observed by Zhang et al. (2005).

4.2 *Comparison with 2×2 pile group*

The response of the single pile is compared to the response of a pile in the group (rotation fixity and translation allowed at pile cap) in terms of peak displacement and peak bending moments (Figure 6 a-f). An interesting observation can be made that in case of nonliquefiable event (0.05g), single pile showed relatively less displacement and lower bending moments compared to the pile group. However, in case of liquefiable events (0.138g and 0.24g), pile group yielded relatively lesser (in the order of 50% reduction) and showed lesser bending moments. This phenomenon of reduced pile displacements and bending moments can be explained by the pile-soil-pile interaction (termed as shadowing effects). In nonliquefiable soils, shadowing effects would lead to the reduced group efficiency compared to the single pile performance and yield higher displacements and bending moments. Similarly, in case of liquefiable soils, as the surficial soil liquefies, chances of shadowing effects are negligible and hence, pile group in

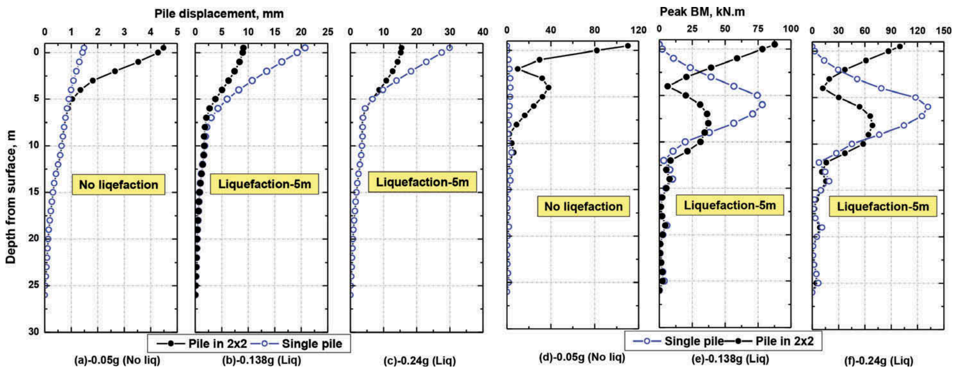

Figure 6. (a) Peak displacement profiles for 0.05g, (b) 0.138g and (c) 0.24g ground motions (d) peak bending moment profiles in case of 0.05g (e) 0.138g and (f) 0.24g seismic events.

liquefiable soils often show relatively less deformations and reduced bending moments in comparison with the single pile.

5 CONCLUSIONS

This article presented the study on the dynamic response of a single pile and 2×2 pile group embedded in liquefiable soil stratum using dynamic BNWF approach. Nonlinear effective stress ground response analyses were performed using the experimentally determined dynamic soil properties reported by the authors elsewhere. Outcomes of the same have been used in SAP 2000 analyses to understand the behavior of foundations under different liquefiable conditions. The results are presented in terms of lateral pile deformation and bending moments.

1. Possibility of plastic deformation of the pile foundations in liquefiable soils and such deformations are significant at the vicinity of the liquefiable and nonliquefiable soils.
2. The peak pile displacements and bending moments increase with the intensity of the input motion (keeping other ground motion parameters constant) despite the same depth of liquefaction for varying ground motion intensities. This is due to the increased free field displacements for high intensity motions.
3. High bending moments are observed at the vicinity of the liquefiable and nonliquefiable soils due to the sharp stiffness contrast.
4. Pile group yields relatively lesser in case of liquefiable soils, when compared to the single pile response due to the absence of shadowing effects.

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