Influence of Excess Pore Pressure Development on Inertial Pile Response

Hande Yumuk and Gurkan Ozden

Abstract The behavior of piles in submerged and liquefiable sandy soils under earthquake loads attracted attention of researchers since 1980s. In this study, behavior of soil surrounding piles in such soils was investigated. In the study, response of the piles under inertial loads was considered, and it was assumed that the soil was in passive terms of during the earthquake. In other words, kinematic and inertial interactions are separated in soil-pile interaction. In the three-dimensional finite element analysis, harmonic loading was applied to the pile head. As a liquefiable soil layer, sandy soil whose liquefaction model parameters were determined at the California Wildlife Test Site was considered. In the analyzes made under different horizontal loads and frequencies, it was determined that the stresses transferred from the pile to the soil can force the soil up to liquefaction by causing excess pore water pressure to change in the soil, and its reflection on the horizontal load–displacement curves (p–y curves) is discussed. The results of the analysis revealed that the loading frequency had some effect on soil behavior and development of excess pore water pressure significantly affected the horizontal load- displacement curves.

Keywords Liquefiable soils · Laterally loaded pile · Excess pore pressure · Pile-soil interaction

1 Introduction

Earthquake response of foundation piles have long attracted attention of researchers since pile damages were observed several times in past earthquakes [1]. Such observations accelerated research and major contributions to the field were already made

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[2, 3]. However, it is still necessary to get more insight on excess pore pressure development around piles in liquefiable soils [4]. In this study, a nonlinear dynamic finite element analysis campaign was pursued to study excess pore pressure regime developing around a pile loaded at the head in saturated liquefiable silty sands. Achieved results indicate that considerable pore pressure develops within the soil surrounding the pile thereby reducing soil resistance by a large margin as a result of inertial interaction.

2 Numerical Analysis Model

Two separate finite element analysis models were established to investigate lateral load–displacement and excess pore pressure pattern around a pile on which cyclic lateral load was applied. The pile diameter was set as 60 cm with an assigned elasticity modulus of 30×10^6 kPa. The 30 m deep first model consisted of a slender pile with a length to diameter ratio of L/B = 15/0.6 = 25. The second model, on the other hand, involved a 1.0 m long pile section where influence of effective stress was taken care of uniform applied vertical stress on the surface of the model, which was consolidated before application of the cyclic load. Lateral load was applied as a traction on pile heads along y-axis. Viscous absorbent boundaries were provided on the sides of both models with plan dimensions of 20mx20m. Fine mesh was applied while discretizing the models with 32,771 ten node wedge elements provided for the long pile model and 82,358 elements for the pile section one Fig. 1a, b. A mesh quality check revealed that over 99% of the elements had Quality Sphere or Quality SICN values larger than 0.5.

(a) (b)

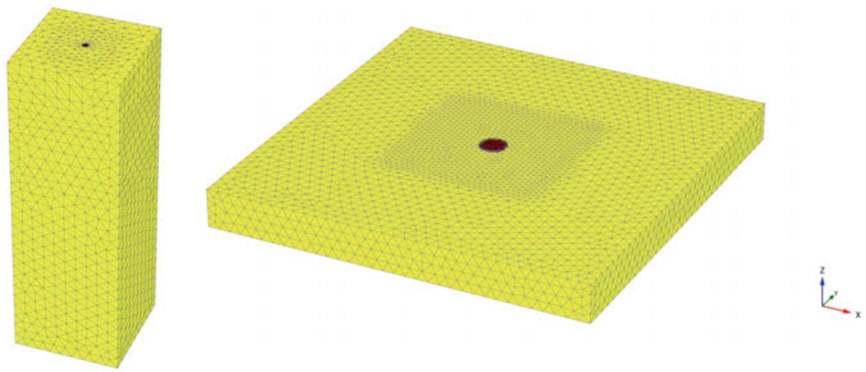

Fig. 1 Finite element model connectivity plots **a** L/B = 25 pile **b** 1.0 m long pile section

Table 1 UBC3D-PLM material model parameters

Parameters	Unit	Value
Young Modulus	kPa	7.28×10^4
Poisson's ratio	–	0.3
Void ratio	–	0.740
Constant volume friction angle	(°)	22
Peak friction angle	(°)	22.765
Cohesion	kPa	1.0×10^{-4}
Elastic shear modulus factor[a]	–	854.6
Plastic shear modulus factor[a]	–	250
Elastic bulk modulus factor[a]	–	598.2
Failure ratio	–	0.811
Densification factor	–	0.2
Post liquefaction factor	–	0.02
SPT-N_{160}	–	7.65

[a]Modulus index values were assigned as 0.5 for elastic, plastic, and elastic bulk factors

3 Soil Characteristics and Material Model Parameters

As the study essentially targets pile response in saturated cohesionless soils Wildlife National Geotechnical Instrumentation Site was chosen as the parent site to obtain material model parameters pertaining to liquefaction analysis. University of British Columbia sand model was the preferred material model due to the availability of calibrated UBC3D-PLM and soil profile parameters for the Wildlife Site in the literature [5]. Soil parameters assigned to the FEM model is given in Table 1. Physical meanings of the parameters and details of the model may be reached out elsewhere [6]. The pile was modeled as an elastic volume element using non-porous material model.

4 Results of Numerical Analyses

Finite element analyses were realized for the full pile length and pile section models. In this respect general trend is to model the pile with its full length and driving the p–y curves along the pile length either through the interface or analyzing the induced stresses inside the soil in the immediate vicinity of the pile [7]. Some researchers, however, indicate that soil resistance is affected by the rotation and flexibility of the pile. It is suggested to restraint displacement of the pile along the loading direction so that pure lateral displacement is achieved [8]. It is considered that pile section model in which the 100 cm pile is only allowed to displace along y-direction would serve this purpose. The effective stress level acting along a pile length is represented by means of an applied surface load. Here, it is aimed to limit the deformations on the interface by keeping the interface strength parameter very low. Soil resistance to the laterally deforming pile was obtained along the interface stress points. Monotonic and cyclic surface traction was applied at the pile head. Loading intensity varied between 25 and 200 kPa in cyclic loading undrained analyses. Cyclic loading frequency was set as 1 and 2 Hz. Monotonic loading analyses, on the other hand, progressed till 300 kPa (i.e. 25~300 kPa corresponding to ≈7~85 kN of lateral point load at the pile head) beyond which soil body collapsed due to excessive pore water pressure and plastic point development (i.e. H = 113 & 141 kN).

4.1 Excess Pore Water Pressure Response

Excess pore water pressure (Δu_e) generation regime because of the lateral displacement of the pile towards and away from the surrounding saturated loose sand is one of the expected outcomes of undrained analyses. Results of monotonic loading tests are presented in Fig. 2a, c for a depth of 3 m below the ground surface. On this figure variation of Δu_e with horizontal strain ε_{yy} is plotted for stress points selected adjacent to the pile as well as 1B and 3.8B away the pile axis parallel to the loading direction. It may be followed on the figure that pore pressure development is associated with dilative soil response adjacent to the pile (Fig. 2a) whereas it is almost completely positive for all loading levels 3.8B away the pile. It is noteworthy that excess pore pressure generation is not highest for larger loads. This trend is related with dilation taking place as loading level increases. At H = 113 and 141 kN, however, negative pressure development ceases at about $\varepsilon_{yy} = 4.5 \times 10^{-3}$ and 7.8×10^{-3}, respectively due to soil body collapse. Influence of head loads less than 42 kN is negligible on excess pore water pressure development.

The figures obtained via undrained monotonic analyses demonstrate excess pore pressure generation potential due to pile displacement. Both dilative and compressive soil response related pressure development tendency increases with loading intensity. Higher positive excess pore pressures are generated 1B away the pile section surface and they quickly diminish at 3.8B distance. At the pile-soil interface dilative soil response is observed as a result of passive wedge development. Maximum induced pore pressure, however, did not induce liquefaction at 1B distance since initial effective vertical stress was $\sigma'_{v0} = 37.3$ kPa as opposed to $\Delta u_{max} = 27.8$ kPa.

Influence of loading frequency and intensity on the achieved results are presented in terms of p′–q plots at a depth of 3 m below the model surface. It may be followed in Fig. 3 that a slight influence of loading frequency (1 Hz and 2 Hz) on deviatoric stress is pronounced in three-dimensional full length pile analysis. A two-cycle analyses showed that 2 Hz cyclic loading with an amplitude of 42 kN (i.e. 150 kPa pile head cyclic traction) resulted in more than 30% decrease in deviatoric stress as compared with the 1 Hz case. Although loading intensities are not the same with the full pile length analyses, pile section runs at 1 Hz and 2 Hz cyclic loading with a magnitude of yielded a similar trend. It is interesting to note that deviatoric stresses are comparable

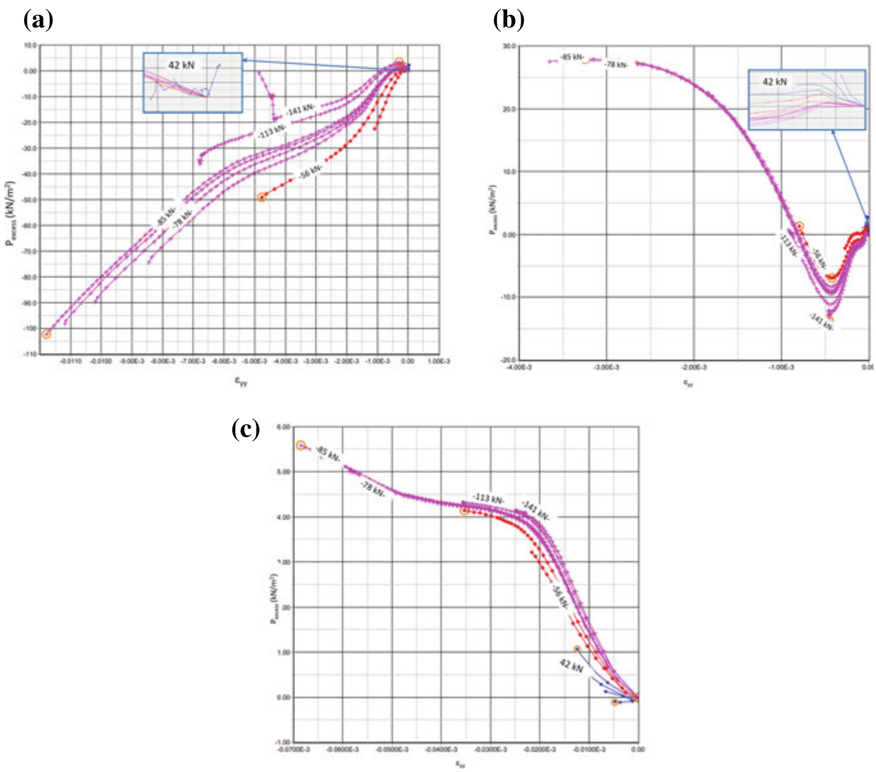

Fig. 2 **a** Excess pore water pressure with respect to horizontal strain adjacent to the pile surface. **b** Excess pore water pressure with respect to horizontal strain 1B away the pile surface. **c** Excess pore water pressure with respect to horizontal strain 3.8B away the pile surface

for both cases probably since same initial effective stresses are achieved in pile section analyses (Fig. 4). A better insight may be captured by applying head loading as a cyclic displacement pattern with a magnitude equal to full pile length analyses. Loading intensity effects on deviatoric and mean effective stress levels are observed in Fig. 5. It is clear that excess pore water pressure generation reduces shear strength especially with lower loading intensities (i.e. 25 kPa and 75 kPa) where a steady decrease towards failure takes place. At larger loading levels, however, dilation takes place at a certain level and shear strength recovery occurs. Dilative tendency quickly disappears with the distance from the pile surface.

4.2 Load–displacement Curves

The load–displacement curves, namely, p–y curves are derived for drained and undrained monotonic loading cases to form backbone curves that would degrade

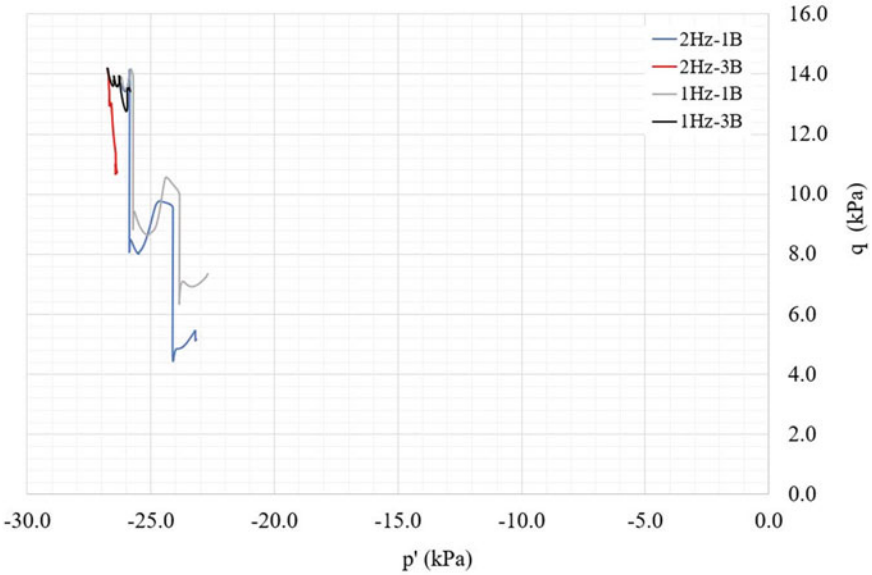

Fig. 3 Variation of deviatoric stress with mean effective stress in full pile length analyses (150 kPa loading amplitude at 2 Hz & 1 Hz frequency)

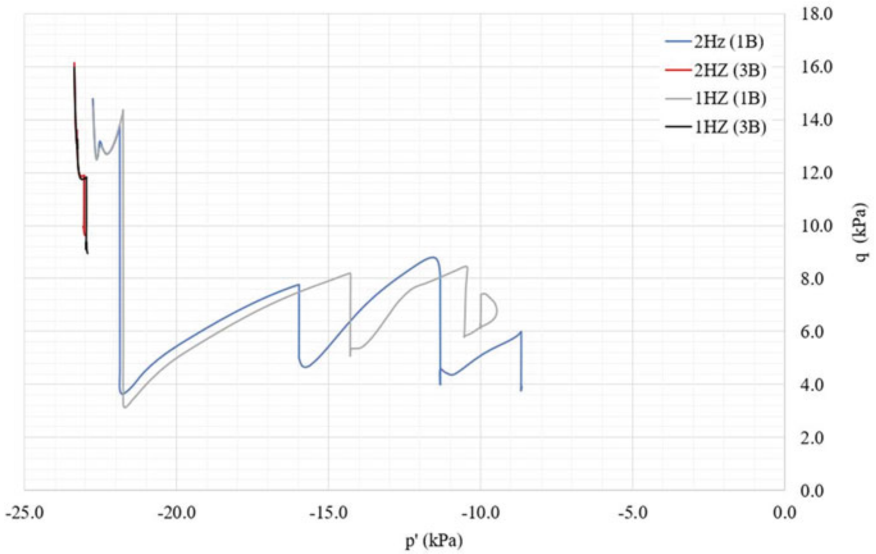

Fig. 4 Variation of deviatoric stress with mean effective stress in pile section analyses (200 kPa loading amplitude at 2 Hz & 1 Hz frequency)

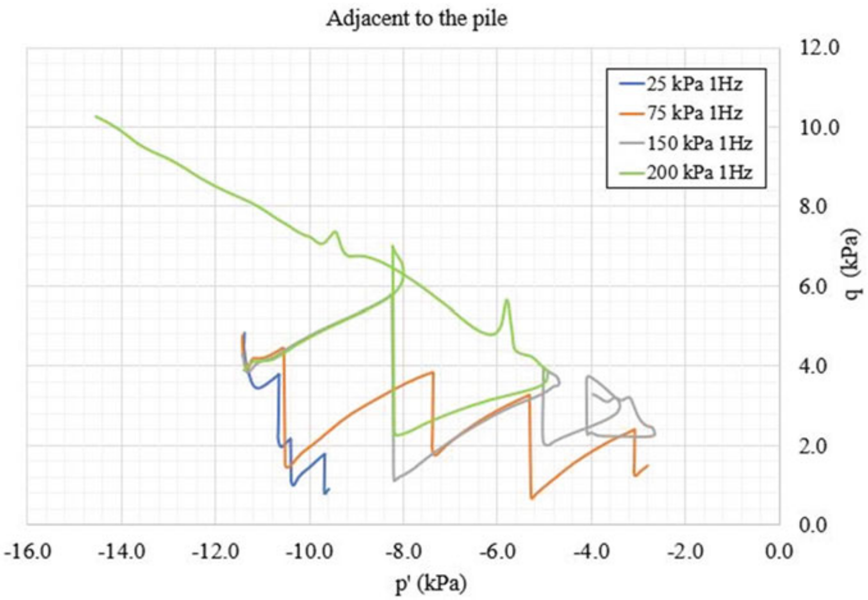

Fig. 5 Influence of loading intensity on variation of deviatoric stress with mean effective stress in pile section analyses (stress points adjacent to the pile)

as effective stress decreases as a result of excess pore water pressure accumulation under inertial cyclic loading conditions (Fig. 6). One may note that API p–y curve fits the drained p–y curve if API model parameters are adjusted accordingly. Further effort is necessary to obtain a p–y curve model that would work for undrained cyclic inertial loads.

5 Conclusions

Finite element analyses results presented herein show that UBC3D-PLM model can be utilized in soil-pile interaction research that targets excess pore pressure accumulation in very loose sandy soils if care is spent on the established numerical model since UBC model may be quite sensitive to boundary value problems. Pile section analyses are found to be applicable to the inertial cyclic loading cases. Full pile length and pile section analyses yielded similar but not the same results. Although spatial redistribution of excess pore water pressure can be handled in pile section model, high gradients parallel to the pile surface along the interface cannot be handled for the time being. As a result of this generated excess pore water pressures are less and soil strength reduction is not that sharp in 3D full pile length model. Both full length and pile section models showed that high pressure gradients take place at the soil-pile interface and dilative response occurs at larger loading amplitudes. Presented

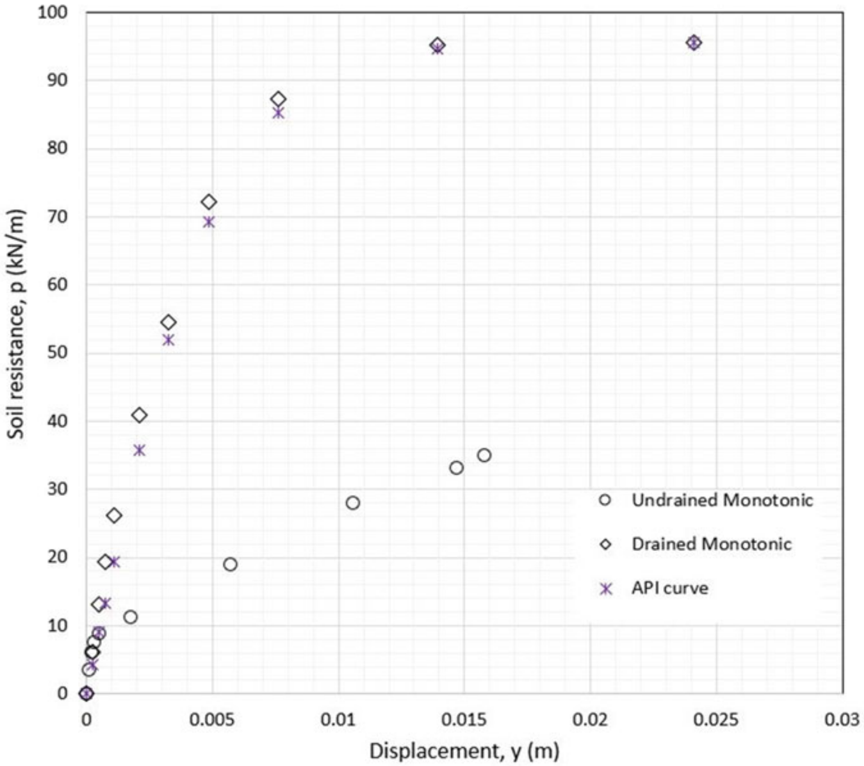

Fig. 6 p–y curves for monotonic drained and undrained loading cases (pile section analyses that correspond to a depth of 3 m below the surface)

p′–q plots and p–y curves belong to a case where soil depth is 3 m below the model surface. Findings of the ongoing research will be shared in future publications.

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