# A parametric study on seismic performance of piles embedded in two-layered liquefiable ground

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ABSTRACT
In this paper, fully coupled three-dimensional dynamic analysis is carried out to investigate the dynamic behaviour of pile foundations in liquefied ground. The critical state bounding surface elastic-plastic constitutive model, Dafalias and Manzari (2004) is used, while a fully coupled (*u-P*) formulation is employed to analyze soil displacements and pore water pressures. Furthermore, variation of permeability coefficient during liquefaction is taken into account. Seismic response of a single pile in liquefiable soil layers is compared with the results of a centrifuge test, and it is found that there is a good agreement between the test results and the computed data. Then, the verified model is used for a parametric study by varying pile length, thickness of liquefying soil layer, relative density of liquefying soil layer and frequency of input motion. The parametric study is carried out for a single pile embedded in a two-layered ground: the upper layer is liquefiable while the lower layer is not. In general, the parametric study demonstrates that thickness of liquefying soil layer and frequency of input motion are the most critical parameters which considerably affect pile performance in liquefied grounds. Therefore, these parameters should be of higher concern during designing pile foundations in liquefiable soil strata.

RÉSUMÉ
Dans ce document, l'analyse dynamique tridimensionnelle entièrement couplée est effectuée pour étudier le comportement dynamique des bases de pile en sol liquéfié. Le modèle d'élastique-plastique extérieur d'état critique, le Dafalias et Manzari (2004) constitutifs de bondissement est employé, et en addition, une formulation entièrement couplée est utilisée pour analyser des déplacements de sol et des pressions d'eau interstitielle. En outre, la variation du coefficient de perméabilité pendant la liquéfaction est prise en considération. La réponse séismique d'une pile simple dans des couches liquéfiables de sol est comparée aux résultats d'un essai de centrifugeuse, et il est possible de constater qu'il y a une bonne concordance entre les résultats d'essai et les données calculées. Par la suite, le modèle vérifié est employé pour une étude paramétrique en variant la longueur de pile, l'épaisseur d'une couche de liquéfaction de sol, la densité relative d'une couche de liquéfaction de sol et la fréquence du mouvement d'entrée. L'étude paramétrique est effectuée pour une pile simple incorporée dans un sol à deux couches : la couche supérieure est liquéfiable alors que la couche inférieure ne l'est pas. Généralement l'étude paramétrique démontre que l'épaisseur d'une couche de liquéfaction de sol et la fréquence du mouvement d'entrée sont les paramètres les plus critiques qui affectent considérablement la performance de piles en régions liquéfiés. Par conséquent, ces paramètres devraient être davantage considérés pendant la conception des bases de pile dans les strates liquéfiables des sols.

## 1 INTRODUCTION

Loose saturated granular soils such as sand are mostly susceptible to liquefaction in which soil loses its shear strength; this results in large lateral deformations and ground failure. Liquefaction phenomenon was reported as the main cause of damage to pile foundations during the major earthquakes such as Alaska, 1964, Loma-Prieta, 1989, Hyogoken-Nambu, 1995 (Kramer 1996). In recent decades, a wide range of centrifuge and shaking table tests and also various numerical methods have been employed in order to provide better insights into the dynamic behaviour of pile foundations in liquefiable soils. These researches can be divided into three categories: field observations, laboratory tests, and numerical models.

*Field observations.* These studies mainly investigate the distribution of the failure patterns, settlement and lateral displacement of piles. Some papers discussing field observation of piles and superstructure failure patterns during past earthquakes are listed for reference: Hamada (1992), Mori et al. (1994).

*Laboratory tests.* These studies include dynamic centrifuge tests and also shaking table tests of pile-supported structures in which seismic response of pile, soil and superstructure are investigated. Wilson et al. (1998) and Liu and Dobry (1995) conducted a series of centrifuge tests on single piles and pile groups located in liquefiable soils in order to observe the p-y behavior of piles embedded in liquefying sands. Some of the recent papers that investigated dynamic behavior of pile foundations in liquefiable soils using shaking table tests are listed for reference: Abdoun and Dobry (2002), Suzuki et al. (2005), Han et al. (2007).

*Numerical models.* Numerical studies consist of one, two and three dimensional modeling of soil-pile-superstructure systems. Since two and three dimensional numerical modeling are computationally complex and time-consuming, most of the researchers and designers prefer to use one-dimensional Winkler method. Liyanapathriana and Poulos (2002) modified this method to take the liquefaction of surrounding soil into account during analyzing process. However, it is obvious that

Winkler models are not able to simulate the prototype system accurately because it is difficult to estimate the accurate values for the springs and dashpots coefficients which considerably change over time. Uzuoka et al. (2007), Cheng and Jeremic (2009) used three dimensional finite element methods in order to simulate piles in liquefying soil layers.

## 2 NUMERICAL MODELING PROCEDURE

Due to the fact that three-dimensional models are able to simulate most of the observed phenomena more accurately than that of one-dimensional models, in this study three-dimensional dynamic analysis is used.

### 2.1 General formulation

In this study, a *u-P* fully coupled formulation is employed in which balance for the soil–fluid mixture, momentum balance for the fluid phase, and finally mass balance for the whole system of soil and fluid are satisfied. This formulation captures the movements of the soil skeleton ($u$) and the change of the pore pressure ($P$). Using the finite element method for spatial discretization, the $u$–$P$ formulation is as follows:

$$M\ddot{U} + \int_V B^T \sigma' \, dV - QP - f^{(s)} = 0 \qquad [1]$$

$$Q^T \dot{U} + HP + S\dot{P} - f^{(p)} = 0 \qquad [2]$$

Where $M$ is the mass matrix, $U$ is the solid displacement vector, $B$ is the strain-displacement matrix, $\sigma'$ is the effective stress tensor, $Q$ indicates the discrete gradient operator coupling the motion and flow equations, $P$ is the pore pressure vector, $S$ is the compressibility matrix, and $H$ is the permeability matrix. The vectors $f^{(s)}$ and $f^{(p)}$ include the effects of body forces, external loads and fluid fluxes.

Numerical integration of the above-mentioned equations and description of the finite element model is performed using parts of OpenSees (Open System for Earthquake Engineering Simulation) framework, a powerful tool for numerical simulation of nonlinear structural and geotechnical systems.

### 2.2 Material model

The critical state two-surface plasticity model developed by Dafalias and Manzari (2004) is used. The most striking feature of this model is its capability to utilize a single set of material parameters for a wide range of void ratios and initial stress states for the same soil. It should be noted that initial stress states, void ratio and fabric evolve through all stages of loading. This model possesses 15 parameters which have been calibrated for Nevada sand by Shahir (2009).

In addition to this, variation of permeability coefficient has been considered in numerical modeling of liquefiable layer. The formulation for the variation of permeability suggested by Shahir (2009) is employed. In this formulation, a direct relationship between the permeability coefficient and excess pore water pressure ratio ($r_u$ : the ratio of the difference of current pore pressure and hydrostatic pore pressure over the initial effective vertical stress) has been proposed.

### 2.3 Finite element model

In this research, soil layers are modeled by cubic eight-node elements with u-P formulation in which each node has four degrees of freedom: three for soil skeleton displacements and one for pore water pressure. Pile is modeled by beam-column elements which have six degrees of freedom for each node: three for displacements and three for rotations. A lumped mass on the pile head represents the superstructure. The finite element mesh is presented in Figure 1. Due to the symmetry of the model, the model is halved at the line of symmetry along the center-line of the pile, and all applied static loads are halved.

Boundary conditions are set in the following way:

- Base of the mesh is fully fixed in all directions.
- At the side planes, parallel to the excitation direction, nodes are restrained from movement in the y direction, and at the ones, perpendicular to the excitation direction, the nodes at equal depths are constrained to have equal displacements in the x direction to simulate free-field ground motion.
- All other internal nodes are free to move in any direction.
- Pore water pressures are free to develop for all nodes except the ones at the ground surface.

Simulations are carried out in three loading stages. In the first stage of loading where pile elements are not installed, self-weight, including both the soil skeleton and the pore water weight, are applied on soil elements. In this stage the initial stress state, void ratio and soil fabric evolve. These values are used as initial values for the next stage of loadings. The second stage includes pile installation and application of its self-weight and the superstructure weight. Then, at the final stage, an acceleration time history is applied to the model as an input motion, and dynamic analysis are performed for the superstructure-pile-soil system.

## 3 EVALUATION OF THE NUMERICAL MODEL

Results of a centrifuge test on pile foundations are used to demonstrate the capability of the numerical model for reliable analysis of piles under dynamic loading. For this purpose, the dynamic centrifuge test of pile-supported structure in liquefiable sand performed by Wilson et al. [4]

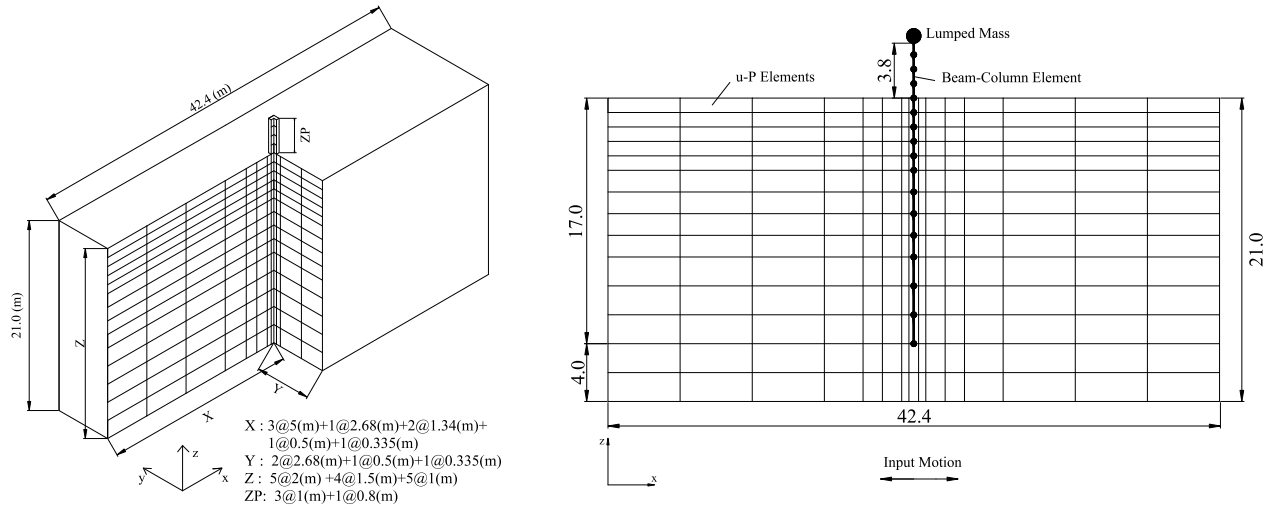


Figure 1. Finite element mesh

is simulated. Figure 1 shows the applied finite element mesh.

In the centrifuge test, soil profile consists of two horizontal layers of saturated, fine and uniformly graded Nevada Sand ($D_{50}$=0.15 mm, $C_u$=1.5). The lower dense layer (Dr=80%) is 11.4 m thick, and the upper medium dense layer (Dr=55%) is 9.1 m thick at prototype scale. Furthermore, the single pile is equivalent to a steel pipe pile with a diameter of 0.67 m and wall thickness of 19 mm at the prototype scale. The pile is extended 3.8 m above the ground level and carries superstructure load of 480 kN; the embedded length of pile is about 16.8 m. The container is filled with a hydroxyl-propyl metyl-cellulose and water mixture whose viscosity is about 10 times greater than pure water. The superstructure-pile-soil system was spun at a centrifugal acceleration of 30g while excited horizontally at the base with Kobe 1995 acceleration record which is scaled to 0.22g. According to the laws of centrifuge modeling, permeability coefficient is three times greater than the value at the prototype scale (i.e. k=1.815×$10^{-4}$ m/s for Dr=55% and k=1.11×$10^{-4}$ m/s for Dr=80% [15] are used in simulation.).

Time histories of the measured and computed excess pore water pressure ratio at the depths of 1 and 4.5 m are presented in Fig. 2. It is important to note that in this study, excess pore water pressure ratio $(r_u)$ is defined as the ratio of the difference of current pore pressure and hydrostatic pore pressure $(\Delta u)$ over the initial effective vertical stress $(r_u = \Delta u / \sigma'_{v0})$. The results indicate that there is generally a good agreement between measured and computed pore water pressure.

Fig. 3 shows the measured and computed bending moment time histories at two different depths of pile; 1 and 2 m. It can be seen that the results obtained from the numerical model agree reasonably well with the values recorded during the centrifuge test. The maximum bending moment at the depth of 1 m occurs at the time t=3.5 sec at which a sudden increase in pore pressure ratio can be observed. This indicates that larger amount of bending moment develops in the pile by softening of the surrounding soil.

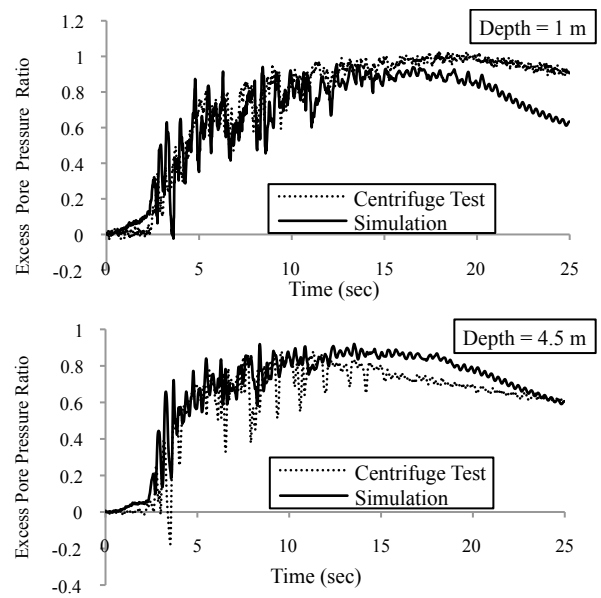


Figure 2. Comparison of numerical time histories of excess pore pressure ratio in the free field at the depths of 1 and 4.5 m with the results of centrifuge test by Wilson et al. (1998).

## 4 PARAMETRIC STUDY

In order to provide better insights into the dynamic behaviour of piles embedded in liquefiable soil layers, a parametric study has been carried out on a soil profile by varying pile length(L), thickness of liquefying soil layer(HL), relative density of liquefying soil layer(Dr) and frequency of input motion(f). In the proposed soil profile, ground is two-layered: the upper layer is liquefiable while the lower layer is not. The finite element mesh used for

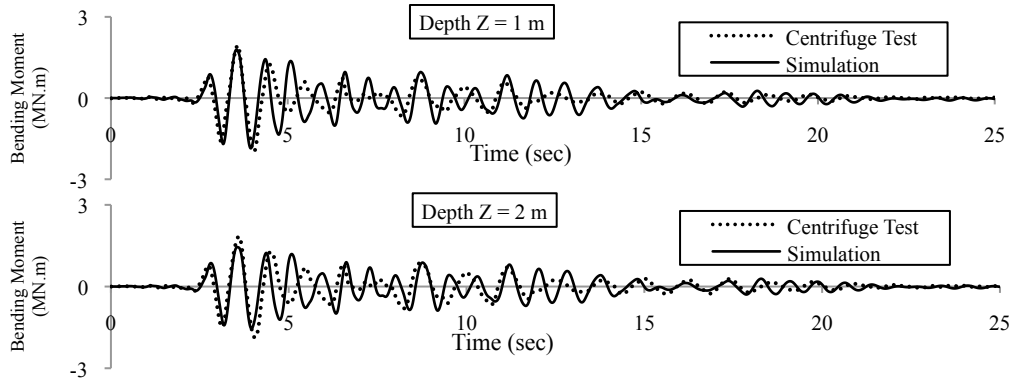


Figure 3. Comparison of time histories of numerically obtained bending moment with the recorded valued during centrifuge test by Wilson et al. (1998).

simulation of the proposed soil profiles is similar to that shown in Figure 1. The concrete pile cross section is assumed to be square with sides (B) of 50 cm, and it is also assumed to remain elastic during the excitation. Unit weight and Young's modulus of the concrete pile are assumed to be 24 (KN/m$^3$) and 3×10$^7$ (kPa), respectively. In all cases, the superstructure is simulated by a single lumped mass with a load of 1000 kN. The input acceleration record used for the analysis is a sinusoidal acceleration time history with 10 seconds duration.

### 4.1 Effect of pile length (L)

In this section, results obtained from repetitive analyses for pile lengths of 15B, 25B and 40B are discussed (B= pile width). As shown in Figure 4, it is concluded that pile length has virtually no effect on maximum lateral displacement of pile during excitation (i.e. from 0 s to 10 s) while significant changes are observed after excitation period (i.e. from 10 s to 25 s). Aristonous et al. (1991) reported that for piles embedded in dry soil layers, pile length has a little effect on pile lateral displacements. However, in this study, it is concluded that pile length significantly affects pile lateral displacements after excitation. From t=10 sec to t=25 sec, inertial forces are minimum not only because the excitation has finished but also because soil layers have liquefied so the kinematic forces, that are predominant, are exerted to the longer pile since the longer pile is in touch with greater amount of the liquefied soil.

Figure 5 shows the maximum bending moment envelops of free-head and fixed-head piles with lengths of 15B, 25B and 40B. In all cases, it can be concluded that pile length has no effect on the place of maximum bending moment, and the maximum value always attained at pile head (for fixed-head pile) or at the depth of about 2 meters (For free-head pile).

### 4.2 Effect of thickness of liquefiable soil layer (HL)

Results obtained from several analyses for thickness of liquefiable layers (HL) of 5 m, 11m and 15 m for a pile length of 40B (i.e. 21 m) are studied. Figure 6 shows maximum lateral displacement of the pile for various thicknesses of liquefiable layers. It is concluded that the thickness of liquefiable soil layer has a little effect on maximum lateral displacement of pile during excitation (i.e. from 0 s to 10 s) while significant changes are observed after excitation period (i.e. from 10 s to 25 s). According to the results, twice increase of the thickness of liquefiable layer leads to about twice increase of the maximum lateral displacement of pile. As mentioned before, after excitation period, piles are intensely under the control of the surrounding liquefied soil, so thicker liquefiable layer considerably affects pile lateral displacements.

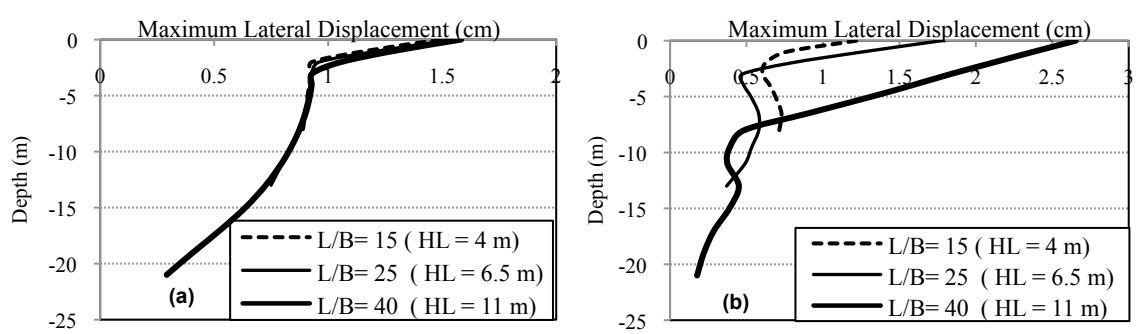


Figure 4. Comparison of pile maximum lateral displacement for different pile lengths (a) during excitation (t= 0 to 10 s) and (b) after excitation (t=10 to 25 s) (L: pile length, B: pile width, HL: thickness of liquefiable layer)

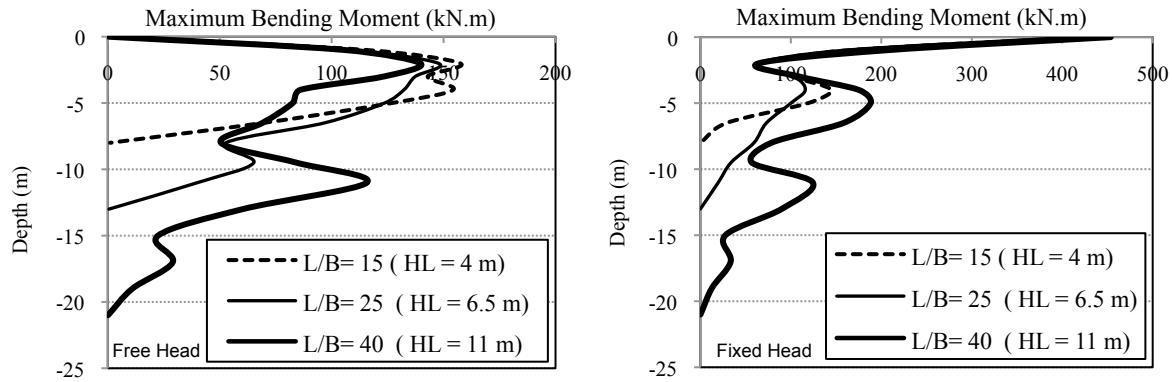


Figure 5. Comparison of maximum bending moment for different pile lengths for free-head and fixed-head pile (L: pile length, B: pile width, HL: thickness of liquefiable layer)

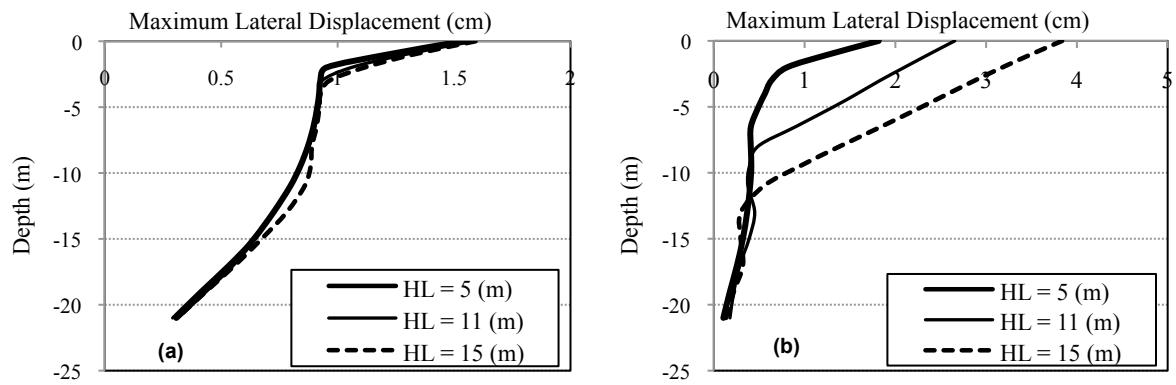


Figure 6. Variation of maximum lateral displacement for various thicknesses of liquefiable layers (a) during excitation (t= 0 to 10 s) and (b) after excitation (t=10 to 25 s)

### 4.3 Effect of relative density of liquefiable soil layer (Dr)

In this section, results obtained from parametric study on three different relative densities; 30%, 40% and 50% for a pile length of 40B are discussed. Figure 7 shows the maximum lateral deflection and maximum bending moment envelops for different relative densities. It should be noted that for a pile embedded in Nevada sand layers, about 10% increase of relative density causes 30% decrease in the maximum value of lateral displacement. Furthermore, it can be observed that 20% increase of relative density causes an average value of 40% decrease in the value of maximum bending moment of the pile. This is due to the increase of liquefiable soil stiffness and smaller values of soil displacement.

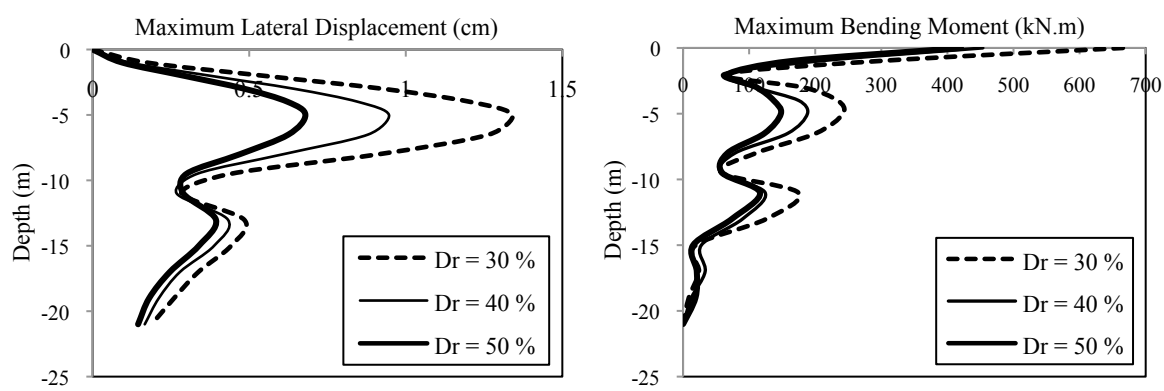


Figure 7. Variation of pile maximum lateral displacement and maximum bending moment by the increase of relative density of liquefiable soil layer

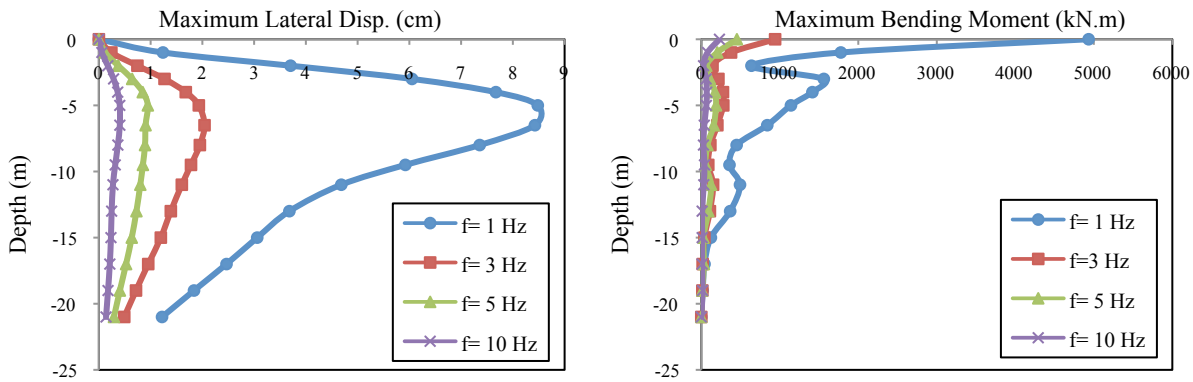


Figure 8. Variation of maximum lateral displacement and maximum bending moment for different frequencies of input excitation

### 4.4 Effect of frequency of excitation (f)

Figure 8 shows pile maximum lateral displacement and pile maximum bending moment time histories for different frequencies of excitation; 1, 3, 5 and 10 Hz. It is noted that frequency has a significant effect on pile response. For example, by increasing the frequency from 3 Hz to 10 Hz, maximum lateral displacement and maximum bending moment decreases about 80%. Since the acceleration amplitude of the input motion is the same in all analysis, displacement amplitude of the soil becomes smaller as frequency increases so small displacement amplitude of the soil has caused relatively small lateral displacement and bending moment of the embedded pile.

## 5 CONCLUSIONS

This study utilizes a verified finite element modeling procedure to analyze the influence of some parameters such as pile length, frequency of input motion, thickness and relative density of liquefying soil layer on dynamic performance of piles embedded in liquefiable grounds. The following results are reached:

1) Pile length and the thickness of the liquefiable layer have a little influence on the maximum lateral displacements during excitation; however, when the input excitation finishes there is a considerable difference among maximum values of the lateral displacement for different pile lengths. It is also shown that pile length has no effect on the place of maximum bending moment.
2) Natural frequency of earthquakes highly affects pile performance in liquefiable soil deposits. It is shown that if the frequency of input motion increases while the amplitude of acceleration remains constant, lateral displacement and bending moment significantly decrease.
3) 10% increase in relative density of liquefiable soil layer results in approximately 20% to 30% decrease in maximum lateral displacement and maximum bending moment.
4) In general, frequency of excitation and the thickness of liquefiable layer has higher effect on pile response compared to the other parameters.

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