Dynamic soil-pile behavior in liquefiable sand overlaid with soft clay

Comportement dynamique sol-pieu dans un sable liquéfiable surmonté par de l'argile molle

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ABSTRACT: A lot of structures are constructed in the vicinity of river streams or sea shores among which bridges, wharves and wind turbines are of more importance. Soil covering these regions is generally constituted of sedimentary loose sand and very soft clay. For this reason, pile foundations are typically used to support the structures and transfer loads to deeper and denser soil. Saturated loose sands which have low relative densities are suspected to liquefy during dynamic loadings like earthquakes. The effect of soft clay overlaying sandy soil is also some sort of questionable during liquefaction. In this study, a 3D model is used to investigate the soil and pile response during liquefaction using FEM method. The constitutive model is based on multi-surface plasticity framework and is capable of predicting sand behavior in undrained condition. Soil response is investigated under some aspects like loading frequency and presence of clayey layer. Pile behavior in terms of pile bending moment is also discussed. It can be concluded that loading frequency and presence of clay layer can have serious effects on pile responses.

RÉSUMÉ : Un grand nombre de structures sont construites à proximité des cours d'eau ou sur les rives de la mer dont les ponts, les quais et les éoliennes sont plus importantes. Les sols dans ces régions sont généralement constitués de sable et d'argile sédimentaire meuble très doux. Pour cette raison, les fondations sur pieux sont généralement utilisées pour soutenir les structures et les charges de transfert aux sols plus profonds et plus denses. Les sables saturés et lâches qui ont de faibles densités relatives sont soupçonnés de se liquéfier sous charges dynamiques telles que les tremblements de terre. L'effet des superpositions d'argile et de sable fin du sol est étudié lors de la liquéfaction. Dans cette étude, un modèle 3Dest utilisé pour étudier le sol et apporter une réponse lors de la liquéfaction en utilisant la méthode des éléments finis. Le modèle constitutif est basé sur le cadre plasticité multi-surface et est capable de prédire le comportement du sable dans un état non drainé. La réponse du sol est étudiée sous certains aspects tels que la fréquence de chargement et la présence de la couche argileuse. Le comportement des pieux en moment de flexion pile est également discuté. On peut en conclure que la fréquence de chargement et la présence d'une couche d'argile peuvent avoir des effets graves sur les réponses de pieux.

KEYWORDS: Soil-Pile interaction, Liquefaction, Dynamic Loading, Multi-Surface constitutive model.

1 INTRODUCTION

Structures constructed at marine areas and river vicinities are endangeded by liquefaction during dynamic loading and specially earthquakes. Some cases of such extensive destruction of structures are observed during earthquakes of Niigata(1964), Alaska(1964), Loma-Prietta(1989) and Kobe (1995). An example of damages to wharf piles caused by liquefaction is shown in Figure 1.

Figure 1.Bending and displacement of pile foundation at Puerto de Coronel Muelle wharf.

The liquefaction mechanisms can be classifed into two main categories: 1) Flow Liquefactin 2) Cyclic Mobility.According to Ishihara (1997), it can be concluded that two types of lateral loading is applied to a pile during an earthquake: 1)Inertial foces 2)Kinematic forces. Inertial forces are applied from superstructure to the pile head and their frequency is a function of structure and the earthquakedominant frequency. Inertial forces are largest before liquefactin and are intensely decreased after the soil is liquefied. Kinematic forces are applied to pile due to the displacement of liquefied soil and they become larger if the ground is inclined or a non-liquefiable layer is overlaid on the liquefiable layer.

Although a lot of research has been carried out to study the liquefaction phenomena and its effect on structures, it can be mentioned that there are some aspects relating to pile response which need to be more clarifiedby utilization of experimental and numerical methods.In the case of experimental studies, some noteable recent works are mentioned briefly. Wilson et al. (1998)accomplished a series of centrifuge tests on piles and their superstructures in liquefiable soil. They observed that sesmic response of soil-pile-superstructure is related to soil type, nature of earthquake and the soil displacements. Other physical test to be noted are Komijo et al. (2004), Yao et al. (2004), Tokimatsu et al. (2005), Dunga et al. (2006), Tamura & Tokimatsu (2005), Han et al. (2007) and Haeri et al. (2012). Some important numerical studies include Finn et al. (2001), Tabesh & Poulos (2001), Miyazaki et al. (2001), Fujita and Finn (2002), Klar et al. (2003), Liyanapathirana & Poulos (2002, 2005), Chang et al. (2007), Bhattacharya et al. (2008), Zhang & Jeremic (2009) and Rahmani & Pak (2011). In many of the numerical studies mentioned above, asoil fluid formulation with uncoupled aproach is used. Furthermore, previously used constitutive models are generaly very simple. These can lead to inacurate results in some cases. Also, some aspects like effect of clay layer and some pile properties related to Offshore Wind Turbines are less discussed. So in this study, it is intended to cover some of these issues.

2 CONSTITUTIVE MODEL AND NUMERICAL FORMULATION

This plasticity model (Parra, 1996, Yang, 2000) is based on the original framework of Prevost (1985), in which a multi-surface approach is adopted for cyclic hysteretic response (Iwan, 1967, Mroz, 1967). The yield function f (Figure 2) is selected of the following form (Prevost, 1985):

$$f = \frac{3}{2}\left(s-(p'+p'_0)\alpha\right):\left(s-(p'+p'_0)\alpha\right) - M^2\left(p'+p'_0\right)^2 = 0 \quad (1)$$

In the domain of $p' \geq 0$, where $s=\sigma'-p'\delta$ is the deviatorice stress tensor, p' is mean effective stress, p'_0 is a small positive constant such that the yield surface size remains finite at $p'=0$. α is second-order kinematic deviatoric tensor defining the yield surface coordinates and M dictates the yield surface size.

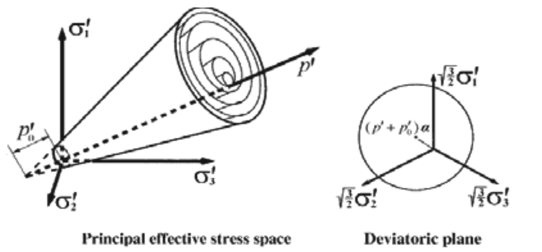

Figure 2.Conical yield surface in principal stress space and deviatoric plane (after Prevost, 1985; Parra, 1996; Yang 2000).

In this model, the contractive, perfectly plastic and dilative phases of Figure 3 are incorporated by developing a new appropriate flow rule.

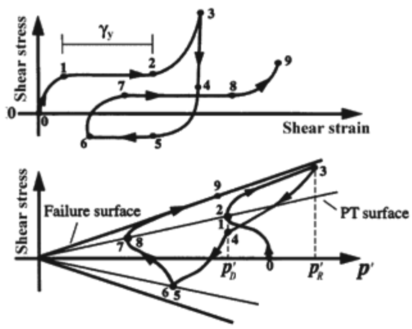

Figure 3.Schematic of constitutive model response showing shear stress, effective confinement and strain relationship [1].

For solving the governing equations of the fully coupled soil-fluid medium, the matrix form of formulation is as follows:

$$M\ddot{U} + \int_V B^T\sigma' dV - QP - f^{(s)} = 0 \quad (2)$$

$$Q^T\dot{U} + HP + S\dot{P} - f^{(p)} = 0 \quad (3)$$

In these equations, M, B, Q, S and H are matrices ofmass, strain-displacement, coupling, compressibility and permeability respectively. Vectors $f^{(s)}$ and $f^{(p)}$dictate the boundary conditions of model including body and surface forces in soil and fluid.

3 MODEL DESCRIPTION AND VERIFICATION

Model is built using OpenSeesPL.Soil dimension is 60x30x35 m in x,y and z directions respectively as shown in Figure 4.

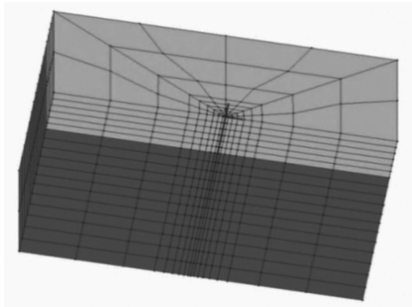

Figure 4. 3D model used in this study

Two general cases for soil layers are considered. For Case I as shown in Figure 5, a sand deposit of 8 m thickness with relative density of 40% is placed over a sand layer with D_r=70% and thickness of 27 m. For Case II in Figure 6, half of the first layer is replaced with a very soft clay. The sand properties are shown in table 1. For very soft clay, shear modulus is 1000 kPa and cohesion is 15 kPa.

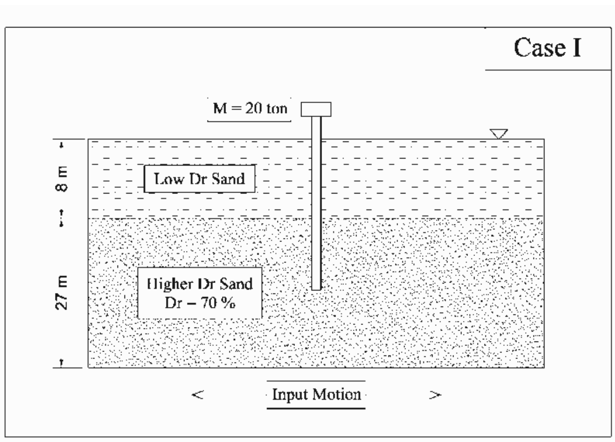

Figure 5. Schematic of Case I

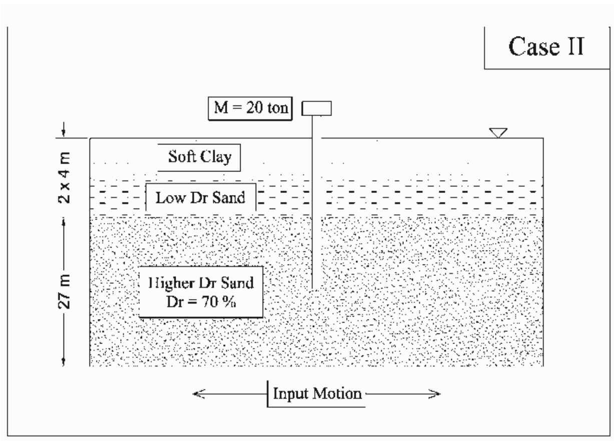

Figure 6. Schematic of Case II

Every node has 4 degrees of freedom(DOF). The first three DOFs represent soil translation in x,y and z directions and the 4th DOF is for pore water pressure. Base nodes are fixed in all directions. Pore pressure degree of freedom is fixed in ground surface to let the water to drain at this region and is open to change in other nodes. Side nodes perpendicular to base motion direction are fixed in this direction and are open parallel to excitation direction. Side nodes parallel to base excitation are constrained perpendicular to excitation direction and free to move in this way. To prevent sides of the model from reflecting dynamic waves, large elements are put in these regions.

Pile properties are selected as follows: diameter = 1 m, length = 21, length above surface = 1 m, thickness = 1 cm, Modulus of Elasticity = 30 GPa, pile head is free to rotate and the material behavior is elastic.Pile elements are connected to soil using very stiff beam column elements. Superstructure mass at top of pile is 20 ton.

Input motion is in the form of sinus wave with PGA of 0.15g and frequencies of 2 and 5 Hz with duration of 10 sec and is applied to the base of the model.The response of system is calculated for a 15 sec time period.

To verify the suitibility of model in reproducing logical responses, a centrifuge test carried out byWilson et al.named Csp3 for Event Jwas selectedand the responses of excess pore pressure and bending moments are compared.

As shown in Figures 7and 8,results of model for excess pore presure ratio and pile bending moment are in relatively good agreement with those of centrifuge tests. So the model is capable of predicting soil behavior under dynamic loading as well as pile responses.

Table 1.Sand parameters used in this study

MODEL PARAMETER	D_R = 75 %	D_R = 40 %
REFERENCE SHEAR MODULUS (KPA)	13x10^4	9x10^4
REFERENCE BULK MODULUS (KPA)	26x10^4	22x10^4
FRICTION ANGLE (DEGREE)	36.6	32
PT ANGLE (DEGREE)	26	26
CONTRACTION PARA 1	0.013	0.067
CONTRACTION PARA 2	0	0.23
DILATION PARA 1	0.3	0.06
DILATION PARA 2	0	0.27
PEAK SHEAR STRAIN	10 %	10 %
PERMEABILITY (M/S)	6.6x10^{-5}	6.6x10^{-5}

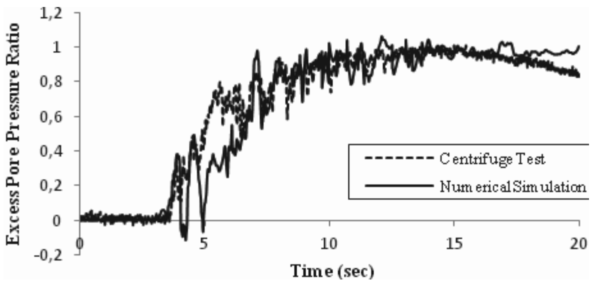

Figure 7.R_u time history at 1 m depth and 6.6 m horizontal distance from pile wall.

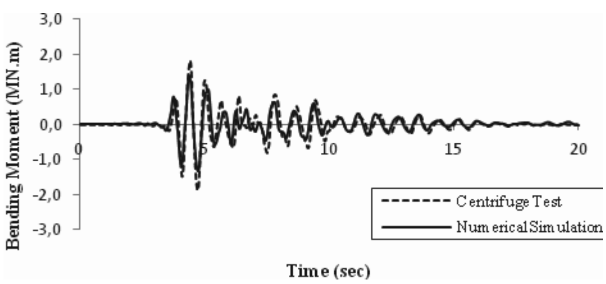

Figure 8.Bending moment time history response of pile at 4 m depth from ground surface.

4 SIMULATION AND RESULTS

Soil and pile response are dicussed in 2 sections. In the first section, R_u is defined as ratio of the excess pore water pressure to the initial effective vertical stress ($R_u=\Delta u/\sigma'_{0v}$). When R_u reachesunity, the soil is totally liquefied and has no more shear strength.

4.1 *Excess pore pressure*

It is observed from Figures 9, 10 and 11 that in the first layer with thickness of 8 m,complete liquefaction happens after a few seconds of excitation which means low $D_r$sand at shallower depths is liquefied under loadings of any frequency, although it is worth mentioning that at higher frequencies, it happens a little later. This can be relatedto oscillation of soil particles to a lower limit at higher frequencies according to Yao et al. (2004).

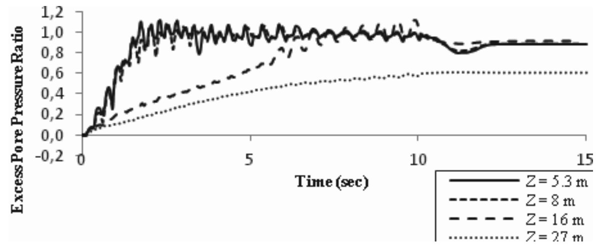

Figure9.R_u time history at 5.3,8 and 27 m depth and 12 m horizontal distance from pile under base motion of 2 Hz frequency for Case I.

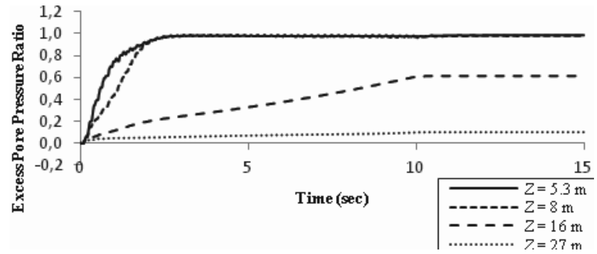

Figure 10.R_u time history at 5.3,8,16 and 27 m depth and 12 m horizontal distance from pile under base motion of 5 Hz frequency for Case I.

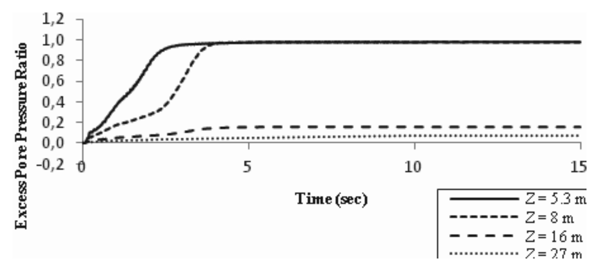

Figure 11.R_u time history at 5.3,8,16 and 27 m depth and 12 m horizontal distance from pile under base motion of 10 Hz frequency for Case I.

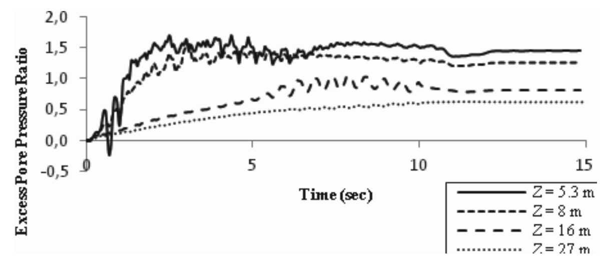

Figure 12.R_u time history at 5.3,8,16 and 27 m depth and 12 m horizontal distance from pile under base motion of 2 Hz frequency for Case II.

By comparing Figures 9 and 12, it can be concluded that the peak quantities of graph inCase II have reached to values upper than unity and also the increase in R_u in this case happens a little faster. This can be rlated to the effect of upper clay layer in Case II which prevents the water to flow outside the ground due to lower permeability. As a result, when a layer of clay is present, liquefaction happens more rapidly and may extend to deeper ground as well as shallower depths.

4.2 *Pile bending moments*

At the beginning of loading, it is observed that the maximum value of bending moment occurs at shallower depths and as the time passes, the place of maximum bending moment moves downward due to reduction in soil strength after liquefaction.

By comparing Figures 13 and 14, it is concluded that before liquefaction occurs or at early times of liquefaction, maximum bending moment for Case II is approximately 100% more than that of Case I, but it has to be said that this large bending moment is a resultoflow shear strength of clay sothat the pile acts like a cantilever beam through this depth and therefore the maximum bending moment increases.

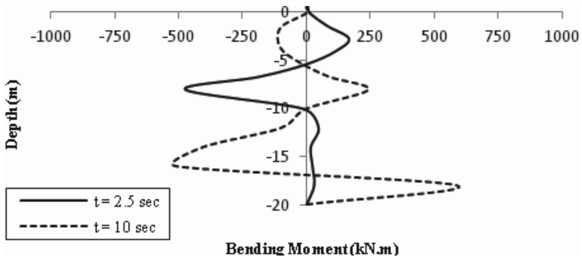

Figure 13. Maximum bending moments in pile at base excitation frequency of 2 Hz for Case I

From Figures13 and 14, it is observed that after liquefaction occurs, pile bending moment is increased up to 37% in Case II in comparison to Case I. In Case II, if the first layer of 8 m is completely constituted of clay, the bending moment is increased 45% in comparison to Case I. This shows that as it was deducted in the previous section, clay layer can affect the deeper sandy layer of soil to liquefy more intensely and as a consequence of decrease in soil shear strength, the soil moves more freely and exerts more pressure to pile. So in this case, kinematic forces increase and cause the pile bending moments to grow significantly.

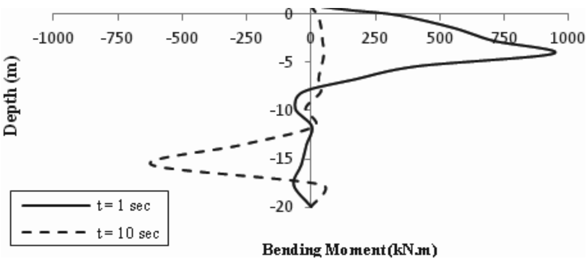

Figure 14. Maximum bending moments in pile at base excitation frequency of 2 Hz for Case II with clay layer thickness of 4 m.

When frequency is increased, it is shown in figure 16 that pile bending moments decrease significantly and also the place of maximum bending moment after liquefaction has moved upward due to less liquefaction of deeper soil at lower frequencies.

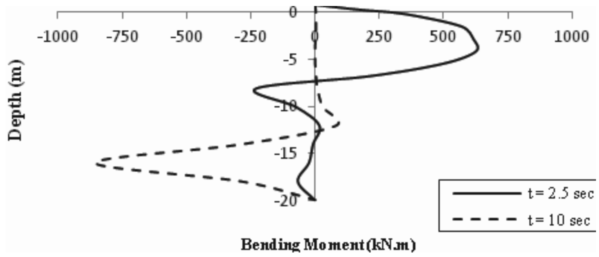

Figure 15.Maximum bending moments in pile at base excitation frequency of 2 Hz for Case I with clay layer thickness of 8 m (No medium sand layer exists).

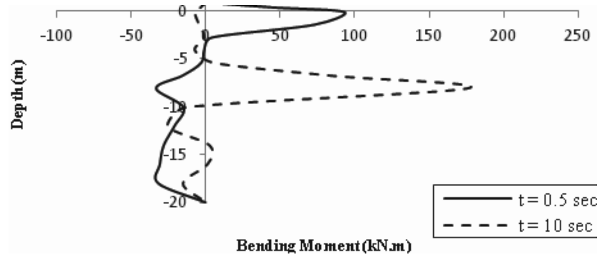

Figure 16. Maximum bending moments in pile at base excitation frequency of 5 Hz for Case I .

5 CONCLUSION

- With increase of input motion frequency, liquefaction does not happen at deeper layers in comparison to lower frequencies. As a result,maximum pile bending moments after liquefaction are decreased in this situation.
- As input motion frequency decreases, liquefaction occurs a little faster.
- If a layer of clay is present over the sandy layer, due to low permeability of clay, water drains at a lower late. So pore pressure is built up under the clay layer and causes the sand to liquefy more intensely.
- At presense of clay, pile bending moments after liquefaction can increase up to 45% in comparison to Case I with no clay layer.

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