Physical Modeling of Soil-Structure Systems Response to Earthquake Loading

Tarek Abdoun[1] · Lenart González[2]

ABSTRACT >> Liquefaction-induced lateral spreading continues to be a major cause of damage to deep foundations. Currently there is a huge uncertainty associated with the maximum lateral pressures and forces applied by the liquefied soil to deep foundations. Furthermore, recent centrifuge and 1g shaking table tests of pile foundations indicate that the permeability of the liquefied sand is an extremely important and poorly understood factor. This article presents experimental results and analysis of one of the centrifuge tests that were conducted at the 150 g-ton RPI centrifuge to investigate the effect of soil permeability in the response of single piles and pile groups to lateral spreading.

Key words Liquefaction, centrifuge tests, permeability, pile groups

1. INTRODUCTION

Liquefaction-induced lateral spreading of sloping ground and near waterfronts continues to be a major cause of damage to deep foundations. In the US, Japan and other countries, buildings, bridges, and other structures supported by deep foundations have been damaged in many earthquakes, with billions of dollars in damages. Permanent lateral ground deformations induce cracking and rupture of piles at both shallow and deep elevations, rupture of pile connections, and permanent lateral and vertical movements and rotations of pile heads with corresponding effects on the superstructure (McCulloch and Bonilla, 1970[13]; Hamada et al., 1986[7]; Mizuno, 1987[14]; Hamada and O'Rourke, 1992[15]; O'Rourke and Hamada, 1992[8]; Youd, 1993[21]; Swan et al., 1996[16]; Ishihara et al., 1996[12]; Tokimatsu et al., 1996[17]; Yokoyama et al., 1997[20]; Tokimatsu, 1999[18]; Dobry and Abdoun, 2001[4]).

While in some cases the top of the foundation displaces laterally a distance similar to that in the free field, in others it moves much less due to the constraining effect of the superstructure, or of the deep foundation's lateral stiffness including pile groups and batter piles. The foundation may be exposed to large lateral soil pressures, including especially passive pressures from the nonliquefied shallow soil layer riding on top of the liquefied soil.

In some cases, this soil has failed before the foundation with negligible bending distress and very small deformation of the foundation head and superstructure (Berrill et al., 1997)[2]; while in others the foundation has failed first in bending and/or has experienced excessive permanent deformation and rotation at the pile heads. The observed damage and cracking to piles is often concentrated at the upper and lower boundaries of the liquefied soil layer where there is a sudden change in soil properties, or at the connection with the pile cap. More damage tends to occur to piles when the lateral movement is forced by a strong nonliquefied shallow soil layer, than when the foundation is free to move laterally and the forces acting on them are limited by the strength of the liquefied soil.

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2. CURRENT PRACTICE AND UNCERTANTIES

Case histories, as well as 1g shaking table and centrifuge model tests, indicate that the effect of lateral spreading on piles can be characterized in first approximation as a pseudostatic, kinematic soil-structure interaction phenomenon, driven by the permanent lateral movement of the ground in the free field. Various foundation analysis and design methods have been proposed, where the soil applies static lateral forces to the pile foundation, either (i) as a function of the relative displacement between the foundation and the free field (p-y approach); or (ii) taking the maximum possible values of these lateral soil static forces which depend on the soil strength within an overall limit equilibrium (LE) method. A third approach (iii) suggested by several Japanese researchers assumes that the liquefied soil is a viscous fluid and hence the lateral soil static forces are a function of the relative velocity, rather than the relative displacement, between foundation and free field (Hamada, 1998[9]; Higuchi and Matsuda, 2002[10]).

There is currently a huge uncertainty associated with the maximum lateral pressures and forces applied by the liquefied soil, which translates into a similar huge uncertainty in the calculated maximum pile bending moments. For example, in the Japan Road Association (JRA) method, the lateral pressure is specified as 30% of the total overburden pressure, while Abdoun et al. (2003)[1] has recommended a constant lateral pressure with depth of 10 kPa. For a range of field conditions involving single piles (but not necessarily pile groups), the JRA and Abdoun method give similar results. A main source of uncertainty is the area over which this pressure is applied in the case of pile groups. Yokoyama et al. (1997)[20] suggests that the value of the lateral pressure must be multiplied for the whole area of the pile group including the soil between the piles, which for a pile separation of 3d (d = pile diameter) may give a lateral force as much as three times greater than if the lateral pressure is applied only to the piles.

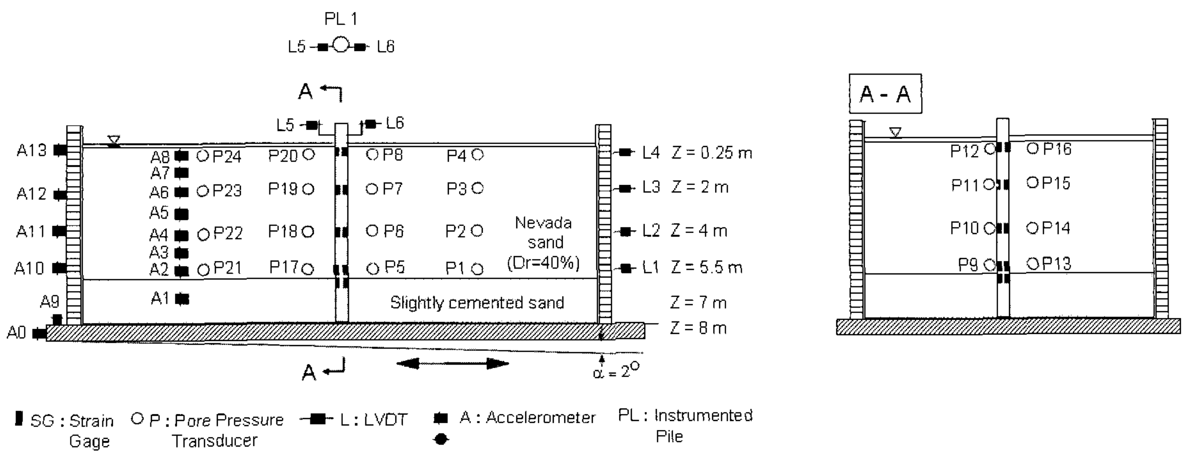
Furthermore, recent centrifuge and 1g shaking table tests (small and full scale) of single piles and pile groups indicate that the permeability of the liquefied sand is an extremely important and poorly understood factor. Several researchers have found out recently that the resistance of the liquefied soil to the movement of an object (pile, cylinder, or sphere) increases as the relative velocity of the object and the soil increases. These results support the theory that the liquefied soil can be modeled as a viscous fluid (e.g. Dungca et al., 2004[5]; De Alba and Ballestero, 2004[3]; Hwang et al., 2004[11]). Dungca et al. conducted small-scaled shaking table tests to study the lateral resistance of a pile subjected to liquefaction- induced lateral flow, where he modeled the pile as a buried cylinder. The results support that the pore fluid migration rate, i.e. the hydraulic conductivity of the soil with respect to the loading rate, is the crucial factor for mobilization of the lateral resistance of a buried cylinder in liquefied soil, because there is less time for the pore fluid to come rushing from the free field to dissipate the negative pore pressures near the pile or other object.

A series of centrifuge tests were conducted the 150 g-ton RPI centrifuge to investigate the effect of soil permeability in the response of single piles and pile groups to lateral spreading. More specifically, six models, simulating a mild infinite slope with a liquefiable layer on top of a nonliquefiable layer were tested in a large laminar box. One model consisted of a single pile (model 1x1-w), other consisted of a line of three piles with a pile cap perpendicular to the direction of the lateral spreading (model 3x1-w) and a third model consisted of a pile group of 2x2 with a pile cap (model 2x2-w). These three models were tested using water as the pore fluid. All models were repeated, using the same fine sand, but saturated this time with viscous fluid (models 1x1-v, 3x1-v and 2x2-v respectively), hence simulating two sands of widely different permeabilities in the field. The importance of the permeability of the liquefied sand can be illustrated comparing the models that simulated the 2x2 pile group. In the model 2x2-w (saturated with water), the pile cap reached a maximum lateral displacement of 7 cm and a maximum bending moment of 55 kN-m in prototype units and then bounced back, while in the model 2x2-v (saturated with viscous fluid) the pile cap reached a maximum displacement of 45 cm and a maximum bending moment of 425 kN-m at the end of shaking, without ever bouncing back. This is a factor of 375/55 ≈ 7 between maximum pile bending moments. Therefore, the uncertainty in lateral soil forces and pile bending moments, related to the poor understanding of the complex behavior of liquefied soils in the vicinity of foundations, can produce maximum lateral liquefied soil forces and pile bending moments varying by factors as high as 3 or 7. It is necessary to reduce this huge uncertainty to more reasonable values in order to develop rational methods of analysis and design of deep foundation subjected to lateral spreads.

3. CENTRIFUGE MODELING OF SINGLE PILE RESPONSE TO LATERAL SPREADS

This paper presents results and analyses of the centrifuge test corresponding to the model 1x1-v of the experimental study mentioned above. Figure 1 presents a sketch of RPI's large laminar box, soil profile, single pile, and instrumentation used in this model. The model height was approximately 0.16 m, simulating under a 50-g level an 8 m prototype soil deposit. The profile consisted of a 6 m layer of loose Nevada sand placed at a relative density of about 40%, overlying a 2 m layer of slightly cemented sand. The pile was embedded into the nonliquefiable layer, simulating an end-bearing pile. The nonliquefiable layer, which consisted of the same Nevada sand but slightly cemented, was placed by dry pluviation and then saturated with water. After 24 hours the liquefiable layer was also placed by dry pluviation and then saturated with a water-metulose solution with a viscosity of about 50 times the viscosity of water, hence simulating a fine sand deposit in the field.

The pile was placed in the model before the soil was pluviated, attempting to simulate a pile installed with minimal disturbance to the surrounding soil, as may be the case when a pile is inserted into a pre-augured hole. The pile consisted in a 0.95 cm diameter polyetherimide rod, simulated at 50-g a prototype pile diameter of 47.5 cm with a bending stiffness (*EI*) of 9000 kN-m^2. After placing the strain gauges at different locations along the pile to measure bending moments, the pile was covered with a thin layer of wax and a soft shrink tube. Then, sand grains were glued to the side of the pile to develop an adequate pile-soil roughness representing the interface between soil and a reinforced concrete pile. The final effective prototype pile diameter was approximately 60 cm. Besides the strain gauges to measure bending moments, LVDTs were installed at the top to measure the pile head displacement. The soil was instrumented with pore pressure transducers and accelerometers, as well as with lateral LVDTs mounted on the rings of the flexible wall to measure soil deformations in the free field. Grids of colored sand were placed at intermediate depths to observe the pattern of soil displacement around the pile at the end of shaking. The model was inclined 2^o to the horizontal, which simulated an inclination of 4.8^o after



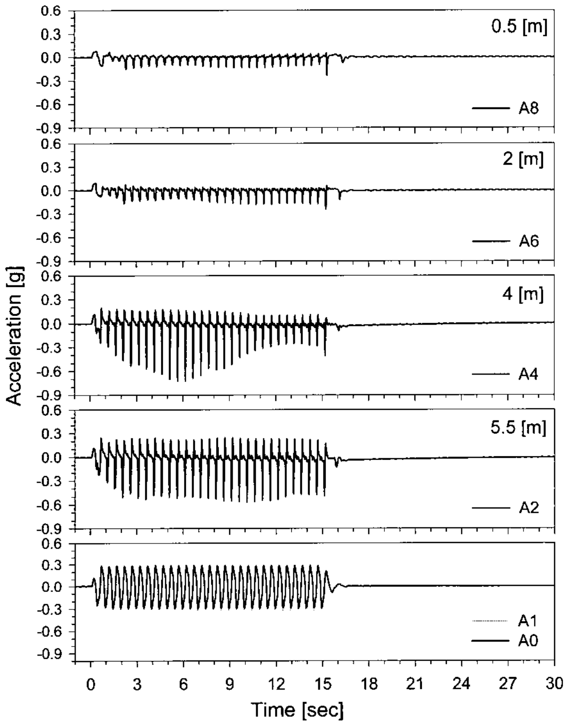
〈Figure 1〉 Sketch of laminar box and instrumentation used in models 1x1-w & 1x1-v depths to observe the pattern of soil displacement around the pile at the end of shaking.

pertinent corrections, thus simulating a mild infinite slope. The models were excited in flight with an input base acceleration (Fig. 2) consisting of 30 cycles of uniform acceleration having a prototype amplitude of around 0.3 g and a frequency of 2 Hz. Experimental results and detailed interpretation of the model are presented in the following section.

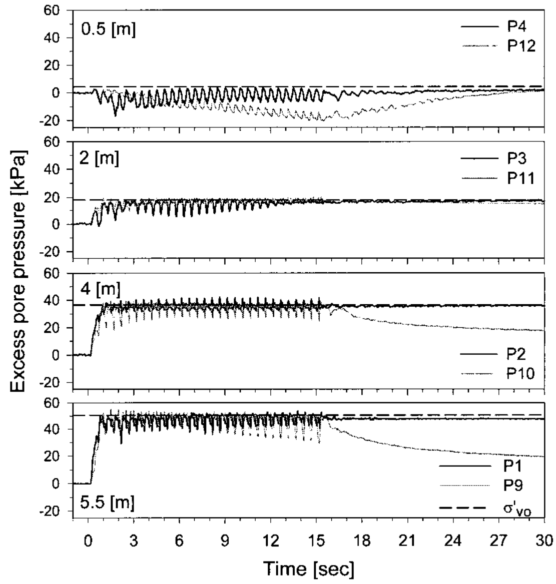
4. EXPERIMENTAL RESUTLS AND ANALYSIS

The input acceleration (recorded) and recorded accelerations in the soil at different depths during shaking are shown in Fig. 2. The corresponding accelerometers were located at reasonable distances from the piles, so these accelerations can be considered as free field data. The acceleration records at the ground surface and at a depth of 2 m decreased significantly after about 1 cycle of shaking due to the liquefaction process and the dynamic isolation of the shallower layers. The acceleration records contain large spikes in each cycle due to the dilative behavior of the saturated loose layer during lateral spreading. The slightly cemented sand acted as a solid layer during shaking, as illustrated by the acceleration records being essentially identically to the input acceleration.

A large number of pore pressure transducers were placed in the model, far away, close and next to the pile, as shown in Fig. 2. These measurements are very important to understand the effect of fluid viscosity in the response of the pile foundations. In the free field, the excess pore pressure records (P1, P2, P3, P4) reveal that the soil liquefied after about one or two cycles of shaking (Fig. 3), in agreement with the trend exhibited by the acceleration time histories. However, near the ground surface the excess pore pressure decreased after a couple of cycles of shaking to values close to zero. Large shear strains developed under low confinement and a slow dissipation process appear to be responsible for this phenomenon. Figure 3 also shows the excess pore pressure measured next to the pile. Negative excess pore pressure developed also near the ground surface (P12); however, in this case the tendency was much stronger, reaching values up to -20 kPa at the end of shaking. The decrease in lateral stress on the down slope side of the pile, and large shear strains with an undrained dilative response of the liquefied soil close to the pile seam to have been responsible for this phenomenon. At larger depths the pore pressure records do not exhibit such a dramatic response. The development of negative excess pore pressure next to the pile generated a large vertical hydraulic gradient, which caused faster pore



〈Figure 2〉 Acceleration time histories

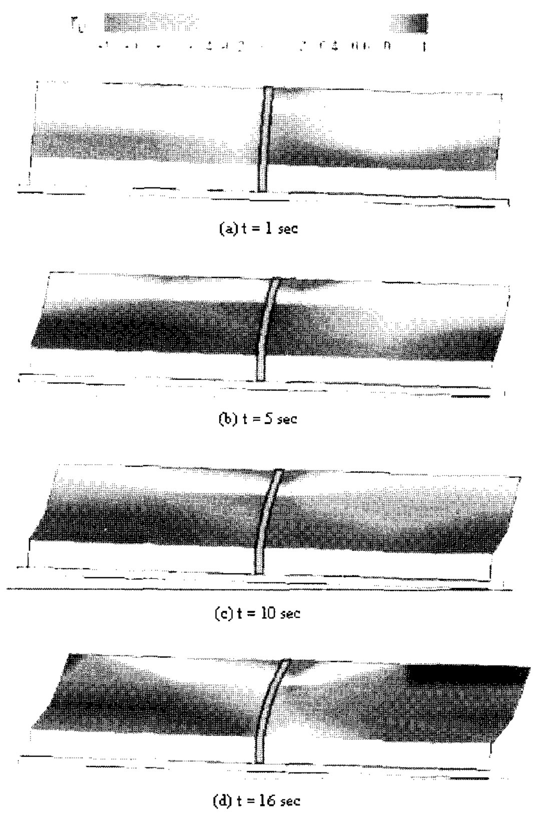


〈Figure 3〉 Excess pore pressure time histories pressure dissipation than a greater distance from the pile or in the free field.

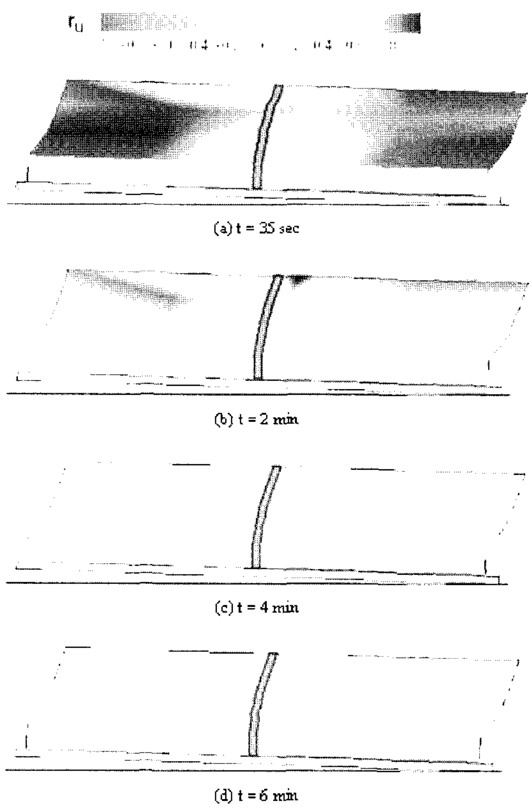
The excess pore pressure records discussed above are very revealing; however, it may not be easy to understand the complex set of data generated by the large number of pore pressure transducers by analyzing time histories of measured excess pore pressures. Therefore, a visual animation was created to further analyze the dynamic response of the model, particularly the pore pressure build-up and dissipation around the pile. The software used was Tecplot 10, a commercial tool with extensive 2-D and 3-D capabilities for visualizing data. The 2-D visualization involves pore pressure ratios and lateral displacement of the soil and pile in the longitudinal direction. These displacements were amplified by a factor of 2 for visual clarity. The field of pore pressure ratio was obtained by interpolating the measurements provided by 20 pore pressure transducers, located far from the pile (P1 - P4, P21 - P24), close to the it (P5 - P9, P17 - P20), and next to the it (P9 - P12).

Figure 4 shows snapshots of the animation at selected time instants during the excitation. After one second the whole loose sand layer was practically liquefied, except near the surface where negative excess pore pressure developed and sustained during all the excitation. These very low pore pressures, particularly around the pile, must have stiffened the liquefied soil in this zone. On the other hand, these negative excess pore pressures created a large vertical gradient close to the pile. Figure 5 shows snapshots of the animation after the shaking stopped. The dissipation process started around the pile, with the free field still being liquefied. The expelled water ended up liquefying the soil on the downslope side of the pile, as shown in Fig. 5b.

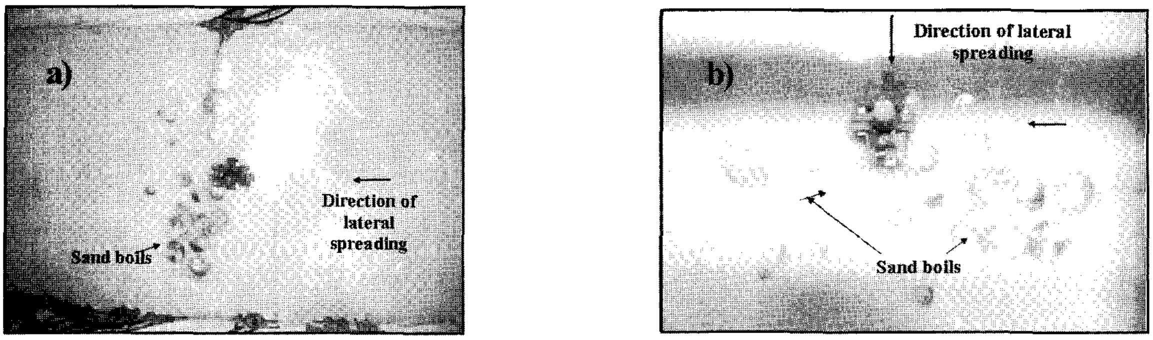
Figure 6 shows pictures of the ground surface, after carefully removing the fluid above it. Unexpectedly, several sand boils developed on the downslope side of the pile, even though the model was fully saturated. This phenomenon validates the hypothesis of a non-liquefied crust close to the pile developed by the negative excess pore pressures. As the water started being expelled, it broke through this crust carrying some of the colored sand that was placed below the surface.



〈Figure 4〉 Short term excess pore pressure ratios and lateral displacements for selected time instants, Model 1x1-v



〈Figure 5〉 Long term excess pore pressure ratios and lateral displacements for selected time instants, Model 1x1-v

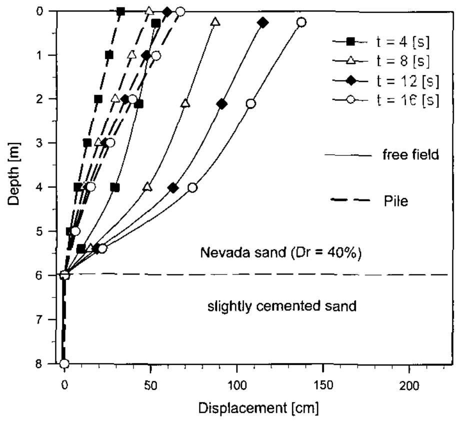


〈Figure 6〉 Sand boils on the downslope side of the pile, (a) far view, (b) close view, Model 1x1-v

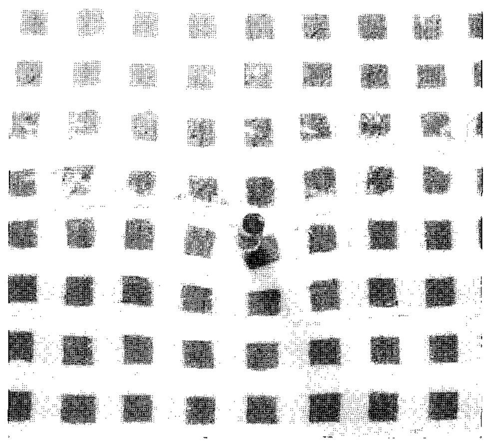
Profiles of the free field lateral displacement were obtained interpolating the values measured with the LVDTs placed in the laminar rings (Fig. 7). As soon as the loose sand liquefied at the beginning of shaking, the deposit started moving laterally downstream, reaching a maximum displacement at the end of the excitation of 140 cm. The pile displacement profiles, without the dynamic component, were obtained as a first approximation through double integration of the interpolated profiles of bending moments along the height, according to the equation:

$$y_p = \iint \frac{M(h)}{EI} dh \qquad [1]$$

where y^p is the pile lateral deformation, EI is the flexural rigidity of the pile, and h is the distance from the slightly cemented layer (height). However, the estimated pile head lateral displacement differed considerably from the measured with the LVDTs L5 and L6, due to the fact that the slightly cemented layer was not able to provide infinite lateral constraint. In a second iteration, the angular rigidity was estimated dividing the bending moment measured at the base of the liquefiable layer by the necessary rotation, so the pile head displacement obtained with equation [1] plus the displacement due to the rotation match the displacement measured with the LVDTs. It was obtained that a value of 8000 kN-m/rad represents very good the angular rigidity of the slightly cemented layer (González et al., 2005)[6]. Finally, considering the deformation by curvature and rotation, the profiles of lateral displacement were obtained (Fig. 7), which increase monotonically during shaking, reaching a maximum displacement of 65 cm at the end of shaking.



〈Figure 7〉 Profiles of free field and pile displacement



〈Figure 8〉 Pattern of soil displacement around the pile (z = 1 m)

As already mentioned, grids of colored sand were placed at intermediate depths to observe the effect of soil permeability in the pattern of soil displacement around the pile. Figure 8 shows a picture of soil condition at 1 m depth, taken after the test. The arrow indicates the direction of lateral spreading. The picture shows a large area of influence due to the presence of the pile, reaching a distance of several times the diameter of the pile.

Bending moment profiles measured at different times in the single pile, after filtering out the dynamic component, are shown in Fig. 9. The pile reached a maximum bending moment of 360 kN-m at the base of the liquefiable layer at the end of the excitation, which should be compared to the model 1x1-w saturated with water (not presented in this article), where the pile bounced back during shaking reaching a maximum bending moment much smaller (120 kN-m).

In order to further investigate the liquefied soil-pile-structure interaction during lateral spreading, the lateral soil resistance (p) was estimated using the simple shear beam theory, according to equation [2], where z is the depth.

$$p = \frac{\partial^2}{\partial z^2} M(z) \qquad [2]$$

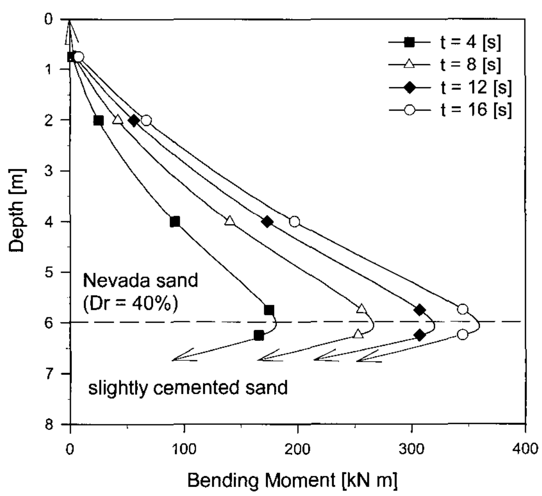
The single pile was instrumented with five pairs of strain gauges along its length, as shown in Fig. 2. The discrete measurements of bending moments along the pile were interpolated using a cubic spline interpolation technique. A cubic spline is perhaps the simplest interpolation of discrete values that can be double differentiated (Wilson, 1998)[19]; however since the spline fits every point exactly, the interpolation is affected by high frequencies (dynamic component) upon differentiation. Therefore, the dynamic component of the bending moment records was filtered out before obtaining the bending moment distributions. The distribution of lateral resistance (p) was obtained by double differentiating the interpolated bending moment distribution with respect to depth.

Figure 10 shows profiles of lateral soil resistance (p) at different instants during the dynamic excitation. The lateral resistance varied between 5 and 20 kN/m below a depth of 2 m. However, near the surface, particularly at a depth of 1 m, the lateral resistance increased considerably during shaking, reaching values of about 50 kN/m.

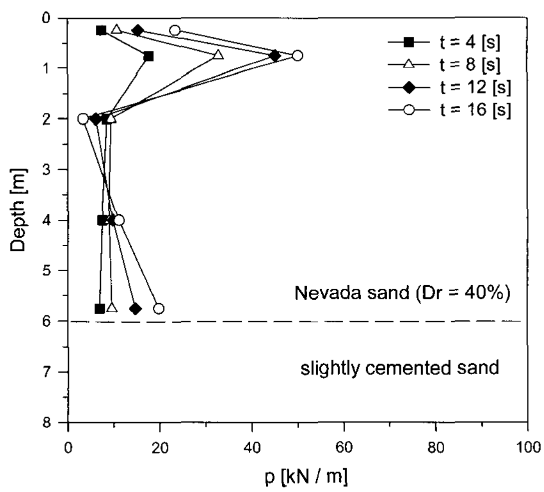
Most probably this negative excess pore pressure near the surface stiffened the soil close to the pile, enabling it to maintain a strong "grip" near the pile head which would explain the high pile displacement and bending moment, as well as the lack of pile rebound. This is consistent with the measured response of the single pile of model 1x1-w, saturated with water (González et al., 2005)[6], where there were no negative excess pore pressures and the pile deformation and bending moments were much smaller. All this highlight a very important effect of the liquefied soil permeability on the pile response subjected to lateral spreading, depending if the dilative effect of the dynamic load on the soil, as well as the decrease in soil lateral pressure downstream of the pile, are able to be canceled out on time by the pore fluid coming from the free field to avoid the development of negative excess pore pressures.

5. CONCLUSIONS

Liquefaction-induced lateral spreading continues to be



〈Figure 9〉 Profiles of bending moment



〈Figure 10〉 Profiles of lateral soil resistance a major cause of damage to deep foundations. Currently there is a huge uncertainty associated with the maximum lateral pressures and forces applied by the liquefied soil, which translates into a similar huge uncertainty in the calculated maximum pile bending moments. This article presents results and analysis of one of the centrifuge tests that were conducted at the 150 g-ton RPI centrifuge to investigate the effect of soil permeability in the response of single piles and pile groups to lateral spreading. The most relevant conclusions are:

- In the model saturated with viscous fluid (model 1x1-v) the pile does not bounce back during base excitation, and bending moments and lateral displacement are much larger than the ones observed in the model saturated with water (model 1x1-w).
- The decrease in lateral stress on the down slope side of the pile, and large shear strains around the pile with an undrained response, seems to have been responsible for the development of negative excess pore pressure near the pile in the model saturated with viscous fluid, particularly near the ground surface.
- This decrease in pore pressure near the pile in model 1x1-v caused a large increase in the liquefied soil resistance around the pile, enabling it to maintain a strong "grip" near the pile head. This large lateral force sustained by the shallow liquefied soil over the pile head until the end of the excitation would explain why the bending moments and pile displacement were much larger than the ones in the model saturated with water.
- The use of colored sand was very useful to visualize the large area of influence around the pile in the model saturated with viscous fluid, which is consistent with the increase in force on the foundation in this case.
- In conclusion, the liquefied soil permeability is an extremely important and poorly understood factor over the deep foundation response subjected to lateral spreading. The phenomenon is complex, including the dilatancy tendency of soil to shear, the decrease in lateral pressure on the down slope side of the pile, and the time the fluid needs to flow from the free field in order to dissipate on time the negative excess pore pressure developed during the excitation near the pile. This study and discussion of model 1x1-v, as well as the comparisons with model 1x1-w and the other models discussed by González et al. (2005)[6] suggest that the pile bending moments and lateral displacements in a silty sand in the field during an earthquake may also be much greater than in clean sand.

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