CENTRIFUGE MODELING OF PILE FOUNDATIONS SUBJECTED TO LIQUEFACTION-INDUCED LATERAL SPREADING IN SILTY SAND

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ABSTRACT :

Liquefaction-induced lateral spreading of sloping ground and near waterfronts continues to be a major cause of damage to deep foundations. Even though various foundation analysis and design methods have been proposed, where the soil applies static lateral forces to the pile foundation, there is currently a huge uncertainty associated with the maximum lateral pressures and forces applied by the liquefie d soil, which translates into a similar huge uncertainty in the calculated maximum pile bending moments. Furthermore, recent centrifuge and full-scale 1g shaking table tests of single piles and pile groups indicate that the permeability of the liquefied sand is an extremely important and poorly understood factor, with a suggestion that the pile bending moments in a silty sand may also be much greater than in a clean sand. Previous centrifuge tests at Rensselaer Polytechnic Institute (RPI) of pile groups and single piles, using the same sand, but with water as pore fluid in one of the tests and viscous pore fluid in the other tests, hence simulating two sands of widely different permeability in the field, have shown significant difference in pile foundation response. This article presents preliminary results of two centrifuge tests conducted at the 150 g-ton RPI centrifuge to continue investigating the effect of soil permeability in the response of 2x2 pile groups to lateral spreading, comparing two techniques to simulate the prototype permeability in the field.

KEYWORDS: Pile foundation, liquefaction, centrifuge, lateral spreading, permeability

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. 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. 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. Even though various foundation analysis and design methods have been proposed, where the soil applies static lateral forces to the pile foundation, 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, 1996) method, the lateral pressure is specified as 30% of the total overburden pressure, while Abdoun et al. (2003) 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 however is the area over which this pressure is applied in the case of pile groups. Yokoyama et al. (1997) suggested 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 full-scale 1g shaking table tests of single piles and pile groups indicate that the permeability of the liquefied sand is an extremely important and poorly understood factor, with a suggestion that the pile bending moments in a silty sand may also be much greater than in a clean sand. This is illustrated by two centrifuge tests of the 2x2 pile group of Fig. 1 conducted at Rensselaer Polytechnic Institute (RPI) by Gonzalez L. (2005), using the same fine sand, but with water as pore fluid in one of the tests, and a viscous pore fluid in the other test, hence simulating two sands of widely different permeabilities in the field. A flexible model container (laminar box) was used in these and other lateral spreading tests, in order to simulate the shear beam free field conditions and to allow development of the lateral spreading. Figure 2 shows the measured bending moment at 6 m depth in the same pile for the two tests, versus pile cap displacement. In the "water" test the pile head reached a maximum prototype displacement of 7.5 cm and maximum bending moment of 62 kN-m and then bounced back, while in the "viscous fluid" test the pile head reached a maximum displacement of 42 cm and a maximum bending moment of 425 kN-m at the end of shaking, without ever bouncing back. This is a factor of 425/62 = 7 between maximum pile bending moments. Therefore, the uncertainty in lateral soil forces and pile bending moments, related to our 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. This certainly constitutes a critical "gap" in our current earthquake engineering knowledge.

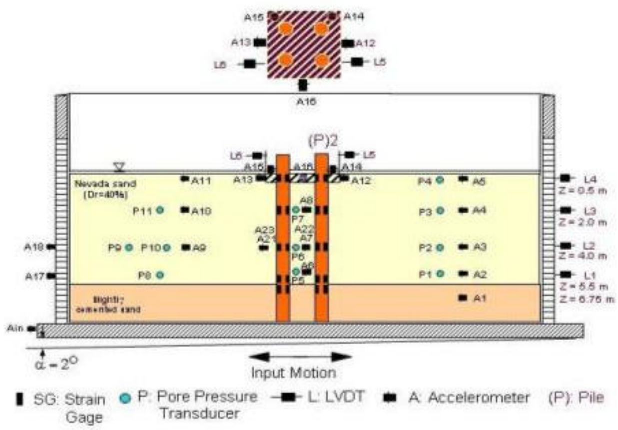

Figure 1 Layout of two centrifuge model tests of 2x2 pile group subjected to base shaking and liquefaction-induced lateral spreading using pore fluids of different viscosities (Gonzalez L., 2005)

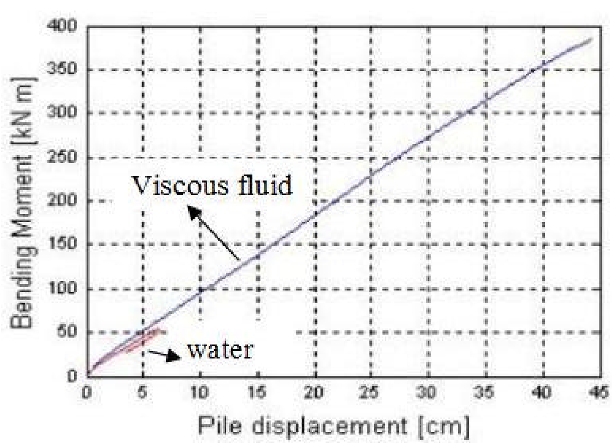

Figure 2 Maximum measured pile bending moment versus lateral displacement of pile head, for 2x2 pile group model of Fig. 1, where water and viscous fluid were used as pore fluid (Gonzalez L., 2005).

Colored sand has been placed near the piles in a number of these centrifuge tests at RPI, in order to further investigate the flow of liquefied sand around the piles. Figure 3 reproduces photos of colored sand placed at mid depth of the liquefied layer for a 3x1 pile group (Gonzalez L., 2005), taken after the test. Figure 3a show clearly that in tests using water as pore fluid, the flow of liquefied sand takes place mostly around the individual piles, affecting only the sand close to the pile itself, and that the movement of the soil beyond this immediate neighborhood of the individual piles is very close to that in the free field. That is the characteristic width perpendicular to the flow determining the flow pattern is the diameter of the individual piles. This would tend to support the hypothesis that the pressure of the liquefied soil acts only on the individual piles and not on the soil in between, contrary to the recommendation by Yokoyama et al. (1997). A completely different liquefied flow pattern is observed when a viscous fluid is used as pore fluid in these centrifuge tests, as shown in Fig. 3b. In these tests, conducted at a 50g centrifugal acceleration, a fluid having a viscosity 50 times greater than water was used as pore fluid, in order to satisfy rigorously the corresponding permeability scaling relation and simulate fine sand in the field, corresponding to the same fine Nevada sand used in the centrifuge experiments. For the 3x1 pile group in Fig. 3b, using a viscous pore fluid, both the soil between piles and the soil to a considerable distance at both sides of the pile group are affected by the flow of liquefied soil. That is, the characteristic width perpendicular to the flow determining the flow pattern when a viscous fluid is used, is the width of the whole pile group, which is significantly greater than the diameters of the individual piles. This tends to support the hypothesis that the pressure of the liquefied soil acts on both piles and soil in between, with significant increase in lateral loads and pile bending moments, in agreement with the recommendation by Yokoyama et al. (1997). In these centrifuge tests conducted by Gonzalez L. (2005) a large number of pore pressure transducer recorded the pore pressure build up during and after the tests. The records show strong negative pore pressure that develops early in the shaking, which is sustained until the end of shaking. This negative pore pressure, most probably caused by both the decrease in lateral stress on the downslope side of the pile, and shearing undrained dilative response of the liquefied soil close to the pile, stiffens the soil close to the piles, enabling it to maintain a strong "grip" on the pile head which would explain the high pile bending moments and pile displacements, as well as the lack of pile rebound and large zone of soil moving together with the piles as the softer liquefied soil in the free field flows not only around the piles, but rather around a large area of stiffened soil around the piles. Finally, in the centrifuge tests using water as pore fluid, it is speculated that the negative pore pressures never develop as the permeability of the soil is so high that water comes rushing fast from the free field and these potential negative pore pressures are instantly dissipated. As the negative pore pressures are dissipated, the soil near the piles expands and increases its void ratio thus becoming more contractive, explaining the softening of the liquefied soil and bouncing back of the pile as it reaches its maximum bending moment. Evidence on pore pressure decrease and stiffening of liquefied soil near piles at large relative displacements has also been observed by other researchers during shaking, in single pile and pile group tests – in some cases with a mass above ground - both in the centrifuge (Wilson et al., 2000; Haig and Madabhushi, 2002) and in 1g full-scale tests (Suzuki and Tokimatsu, 2004). However, in some of those experiments the negative pore pressures and soil stiffening have been a transient rather than a sustained phenomenon. In conclusion, these studies provides evidence of the importance of soil permeability on pile foundations response during lateral spreading for cases when the liquefied deposit reaches the ground surface, and suggests that bending response may be grater in silty sands than in clean sands in the field.

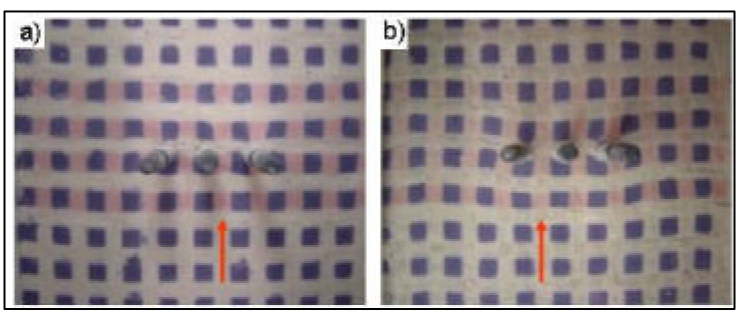

Figure 3 Movement of colored sand around 3x1 pile group, showing flow of liquefied soil around piles. Centrifuge tests with water used as pore flu id in (b) and a viscous pore fluid used in (b). The arrows indicates the direction of lateral spreading (Gonzalez L., 2005)

Since there is a clear difference in pile foundations response observed in centrifuge tests between water saturated and viscous fluid saturated models, Gonzalez M. et al (2007) developed at RPI an alternative way to match the prototype permeability using centrifuge modeling. Instead of using viscous fluid and keeping the same soil gradation, it was proposed to reduce the particle size and use water as pore fluid. Gonzalez M. et al (2007) found out the combination of particle size reduction and soil density in order to mach the prototype permeability and soil compressibility. This article presents preliminary results of two centrifuge tests conducted at the 150 g-ton RPI centrifuge to investigate the effect of soil permeability in the response of 2x2 pile groups to lateral spreading, comparing both techniques to simulate the prototype permeability in the field. These tests were conducted in a large laminar box, simulating a mild infinite slope with a liquefiable layer on top of a nonliquefiable layer.

2. MODEL SETUP

This article presents results of two centrifuge tests conducted to investigate the response of a 2x2 pile group to lateral spreading. These models simulate a 2x2 pile group embedded in a two layer soil system, as shown in Fig. 4. The profile is very similar to the ones used in the previous models conducted at RPI, consisting of a liquefiable layer on top of a nonliquefiable layer. Both slightly inclined models were excited by the same input motion, so the results can be compared. The difference is that Model 2x2-vf was built with clean Ottawa sand and saturated with viscous fluid, whereas Model 2x2-rp was saturated with water and built with Ottawa sand including fine particles, following the reduction of particle technique developed by Gonzalez M. et al. (2007). Both models simulate the same fine sand deposit in the field, with the same permeability. These models simulate a 2x2 pile group connected with a pile cap. The prototype profile consists of a 6 m thick Ottawa sand layer placed at a relative density of about 45%, on top of a 0,625 m thick nonliquefiable cemented layer. The models, inclined 2° to the horizontal (5° after instrumental correction), simulate an infinite mild ground slope. The embedded piles have a prototype diameter (d) of 32 cm and a prototype bending stiffness (EI) of approximately 25000 kN-m^2, and have a spacing between piles of (3d). The setup and instrumentation used in Models 2x2-vf and 2x2-rp are presented in Fig. 4. The models were instrumented with accelerometers, pore pressure transducers and LVDTs. Two piles were instrumented using strain gages, as shown in Fig. 4. Accelerations in the soil and outside the laminar box, excess pore water pressure in the free field and next to the piles, lateral displacement of the soil and the pile cap, and bending moments were measured during the tests. Soil properties used in both models are presented in Table 1, Model 2x2-vf corresponds to the model saturated with viscous fluid and Model 2x2-rp corresponds to the model saturated with water using the reduction of particles technique. At a centrifugal acceleration of 25g the permeability of Model 2x2-rp (25 x 3.86x10^{-4}) is practically the same one of Model 2x2-vf. Both models were excited by 70 cycles of a 50 Hz sinusoidal acceleration with four different amplitudes, as shown in Fig.5. At a centrifugal acceleration of 25g this corresponds to a frequency of 2 Hz and accelerations of about 0.0085g, 0.03g, 0,1g and 0.37g. This input base acceleration was used in order to compare the results with centrifuge tests conducted by Gonzalez M. (2008). The main objective of using increasing amplitudes is to analyze the pile foundation response before and after soil liquefaction takes place.

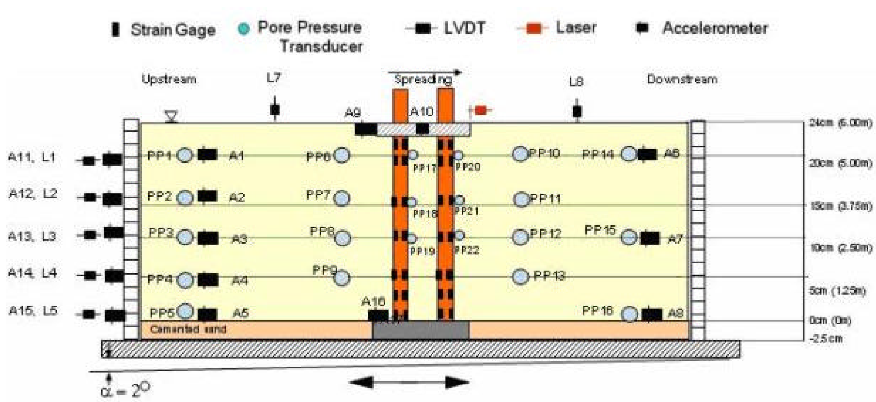

Figure 4 Setup and instrumentation used in models 2x2-vf and 2x2-rp

3. EXPERIMENTAL RESULTS

3.1. Recorded accelerations

The recorded input acceleration and soil accelerations at two different depths during shaking are shown in Figure 5. The corresponding accelerometers were located at reasonable distances from the piles, so these accelerations can be considered as free field data. The acceleration records below a depth of approximately 3.5 m are similar in magnitude to the input excitation during the first 15 seconds. Afterwards, the amplitude of acceleration do not increase in the positive direction. This significantly reduction compared to the input excitation is due to the liquefaction process and dynamic isolation of the shallower layers, which is consistent with the excess pore pressure records. In the negative direction the acceleration records contain large spikes in each cycle due to the dilative behavior of the saturated loose layer during lateral spreading. Even though the spikes were larger in Model 2x2-vf (model saturated with viscous fluid) than in Model 2x2-rp, the records indicate that the soil acceleration in the free field was not significantly affected by the different techniques.

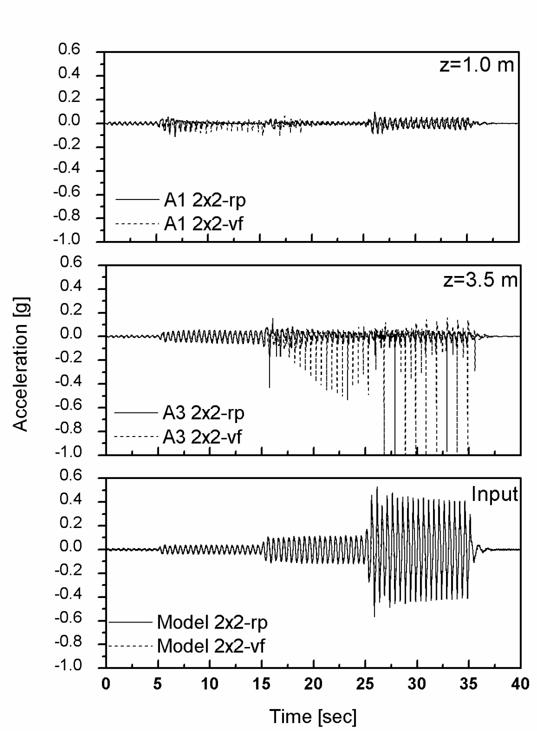

Figure 5 Input acceleration and free field soil acceleration time histories, Models 2x2-vf and 2x2-rp

3.2. Recorded excess pore pressures

A large number of pore pressure transducers were placed in the models, far away, close and next to the pile group, as shown in Fig. 4. These measurements are very important to understand the effect of model preparation technique and the response of the pile foundations. In the free field (Fig. 6), the excess pore pressure records (PP1, PP15, and PP5) reveal that the soil near the ground surface liquefied in the second acceleration phase, after 5 seconds. Below 3.5 m the soil liquefied in the third phase of input acceleration, after 15 seconds, in agreement with the trend exhibited by the acceleration time histories. At that time the excess pore pressure near the ground surface in both tests started decreasing to reach negative values at the end of shaking. Large shear strains developed under low confinement and a slow dissipation process appear to be responsible for this phenomenon. No clear difference in excess pore pressure development is observed between both models.

Figure 7 shows the pore pressure ratio measured next to one of the piles. Next to the pile group the soil did not reach full liquefaction during the tests. Furthermore, negative excess pore pressure developed near the ground surface (PP17) at the end of the shaking process. The decrease in lateral stress on the downslope side of the pile

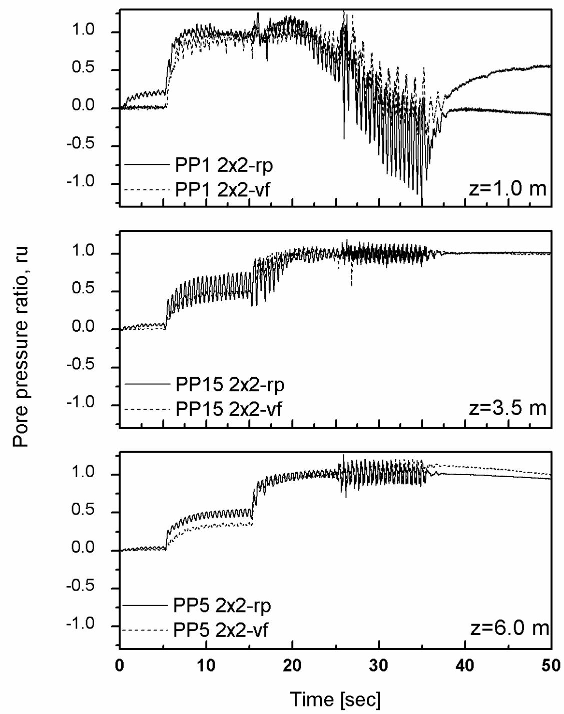

Figure 6 Free field pore pressure ratio time histories, Models 2x2-vf and 2x2-rp

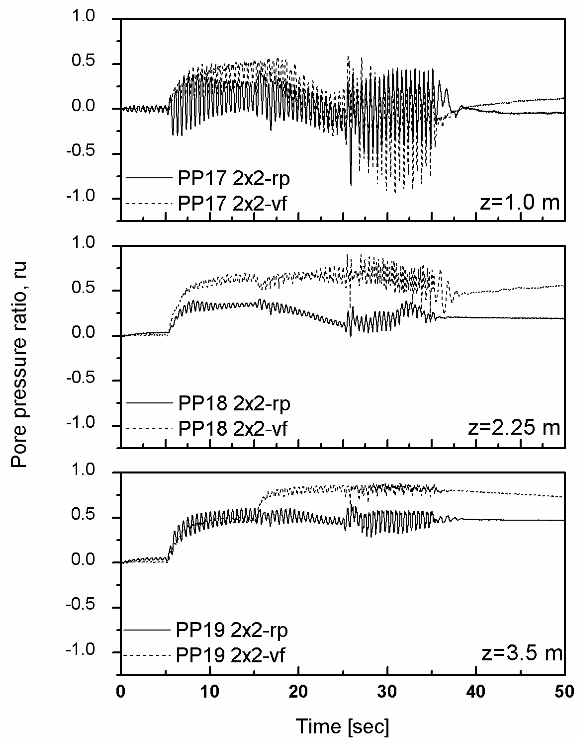

Figure 7 Pore pressure ratio time histories next to the pile group, Models 2x2-vf and 2x2-rp

group, and large shear strains with an undrained dilative response of the liquefied soil close to the piles seam to have been respons ible for this phenomenon. Even though the excess pore pressure development next to the pile group in both models show a similar trend, the excess pore pressure were lower in Model 2x2-rp.

3.3. Recorded soil lateral displacements

Soil lateral displacement in the free field was similar in both models. Near the ground surface the prototype lateral displacement at the end of shaking was 80 cm in Model 2x2-rp and 78 cm in Model 2x2-vf. This excellent agreement in soil lateral spreading strongly support that the particle reduction technique is useful to simulate the field permeability without changing the soil response.

3.4. Recorded pile bending moments

Pile bending moments along two instrumented piles were obtained with the attached strain gages (Fig. 4). Figure 8 shows the pile bending moment profiles at the end of shaking in both models (Model 2x2-vf and 2x2-rp), after filtering out the cyclic component. As expected, the two maximum moments of opposite sign took place at the base of the liquefied layer and at the connection with the pile cap. In both cases the maximum bending moment profiles are quite similar, reaching values at the base of the order of 180 kN-m in Model 2x2-vf and 290 kN-m in Model 2x2-rp. These bending moments are significant large, being consistent with the large bending moments measured in Model 2x2-v (saturated with viscous fluid) conducted by Gonzalez L. (2005) and with the fact that the piles practically do not bounce back during shaking.

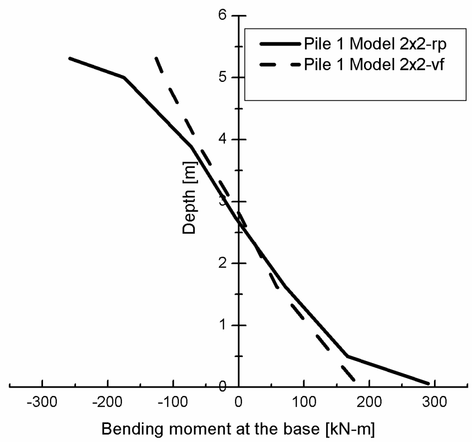

Figure 8 Pile bending profiles at the end of shaking, Models 2x2-vf and 2x2-rp

4. CONCLUSIONS

This article presents preliminary results of two centrifuge tests conducted at the 150 g-ton RPI centrifuge to investigate the effect of soil permeability in the response of 2x2 pile groups to lateral spreading, comparing two methods to simulate the prototype permeability in the field, the first one using viscous fluid (Model 2x2-vf) and the second one using water and a particle reduction technique (Model 2x2-rp). The most relevant conclusions are:

- The excellent repeatability in soil lateral spreading in both models strongly support that the particle reduction technique is useful to simulate the field permeability without changing the soil response in the free field. Furthermore, no significant difference in soil acceleration and excess pore pressure development was observed between both models in the free field, supporting the hypothesis that both modes may be directly compared.
- Near the pile group the soil did not reach full liquefaction during the tests and negative excess pore

pressure developed near the ground surface at the end of the shaking process in both models. The low soil permeability appears to have been responsible that the fluid could not flow fast enough from the free field to dissipate on time the negative increments of excess pore pressure developed in this area in both models. Most probably the low and negative excess pore pressure developed close to the piles, enabling it to maintain a strong "grip" near the pile head, which would explain the large pile bending moment.

- Measured bending moments in both models are similar and larger than the ones observed in previous models saturated with water. The fact that the response of pile foundation in both models is similar would tend to support the use of viscous fluid as a technique to properly simulate the right permeability in the field.
- 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 downslope 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 suggests 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 a clean sand due to the lower soil permeability.

5. ACKNOWLEDGEMENTS

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