4th International Conference on
Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. 1465

BEHAVIOR OF FLOATING PILES IN LIQUEFIABLE SOILS

Ahmet PAMUK [1] and Thomas F. ZIMMIE [2]

ABSTRACT

Centrifuge tests were conducted to study the behaviors of floating single and pile groups (2x2). The pile models were buried in a two-layered soil system, consisting of a non-liquefiable layer at the top, followed by a liquefiable layer, that simulated a fully saturated, granular infinite slope with a prototype inclination of about (α_{field}) 4.8^o where the pile group model were embedded in the non-liquefiable layer and the loose sand layer. The models were tested a 50-g centrifuge acceleration and excited by 30 cycles of a 100 Hz sinusoidal input parallel to the base of the laminar box, with a uniform acceleration amplitude about 13g . The test results, such as acceleration records, permanent lateral displacements, and bending moments and axial forces on the piles, are presented. The results of tests on both floating single pile and pile group foundations indicated that bending moments developed along the piles were controlled by the lateral pressures exerted by the liquefied layer. Measured moments were larger for pile group foundations due to the framing action (or group action) that limited lateral soil deformation caused by the piles in the pile group. A simple limit equilibrium procedure was proposed based on the results from single pile model tests. The proposed analysis predicted the measured bending moments quite well, by using the concept of framing action limiting the lateral deformation in the pile groups.

Keywords: Floating piles, lateral spreading, liquefaction, centrifuge testing

INTRODUCTION

Figure 1 shows the deep foundation system of the Niigata Family Court House (NFCH) building consisted of both floating (Pile-1 in the figure) and end-bearing single piles, both of which were damaged due to liquefaction-induced ground deformation during the 1964 Niigata earthquake (Hamada et al., 1986; Kawashima et al., 1988; Hamada, 1992). A centrifuge test (Model 4, Figure 2) was conducted by Abdoun (1997) using a floating single pile model. The test aimed to study the effect of lateral spreading on a single floating pile embedded in a soil system similar to the soil system that existed at the NFCH site where lateral spreading was the main source of the structural instability that resulted from large ground displacements.

Model 4 indicated that the recorded moments increased with time until they reached a maximum value, and then remained approximately constant until the end of shaking. A simplified limiting equilibrium analysis which considered the floating pile as a cantilever beam embedded in the top nonliquefiable was proposed to calculate the maximum bending moments in Model 4. The analysis predicted that a soil pressure of 9.25 kN/m^2 was uniformly distributed along the pile length before soil fully liquefied (Figure 6). The estimation of the maximum bending moments using this analysis resulted in good agreement with those measured in Model 4.

[1] Florida State University, Panama City, FL-USA; Email: apamuk@pc.fsu.edu
[2] Rensselaer Polytechnic Institute, Troy, NY-USA; Email: zimmit@rpi.edu

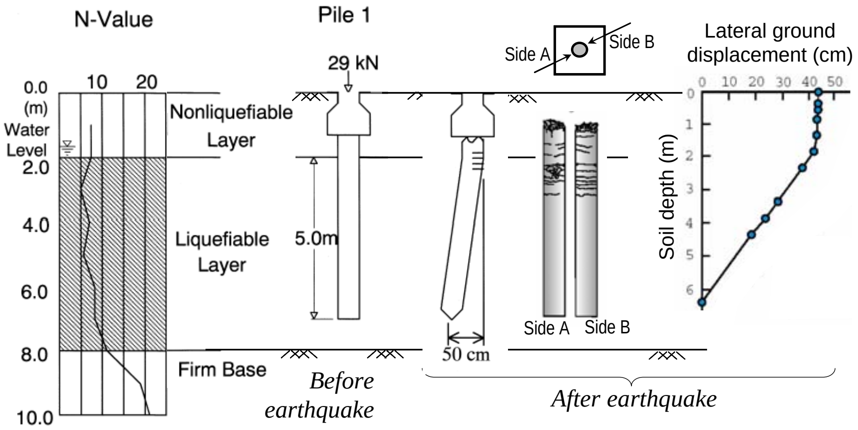

Figure 1. Geotechnical conditions under the NFCH building and damage in floating pile after the 1964 Niigata earthquake (modified after Hamada et al., 1986; Hamada & O'Rourke, 1992; Hamada, 2000).

This paper presents Model 4G (Figure 2) that studied the behavior of a floating 2x2 pile group that was similar to Model 4 in terms of testing parameters. That is, the centrifuge soil profile consisted of a three-layered soil system, with a nonliquefiable sand layer at the top, followed by a loose Nevada sand layer of Dr=40%, and another layer of nonliquefiable sand beneath the loose sand layer (Figure 2). All models simulated a fully saturated, granular infinite slope with a prototype inclination of about (α_{field}) 4.8^o where the pile group model is embedded in the top nonliquefiable layer and the loose sand layer. The test results, such as acceleration records, permanent lateral displacements, and bending moments on the piles, are presented in this chapter. The results from Model 4G are compared with Model 4 and are presented herein. This paper also shows that the limiting equilibrium analysis proposed based on findings of Model 4 can be applied to predict maximum mending moments for pile groups.

TESTING PROCEDURE

The centrifuge testing plan and model setups used, including the location of instrumentation are shown in Figure 2. The centrifuge tests were conducted on two different models, Model 4 (floating single pile) and Model 4G (floating pile group). The laminar box was employed to simulate flexible shear beam boundary conditions for soil in the free-field and allows for permanent lateral deformation of the soil during shaking. Additional details regarding the box can be found at Pamuk (2004). A large in-flight computer-controlled centrifuge shaker was used to produce the base dynamic excitation. Water-saturated Nevada No. 120 fine sand was used as the liquefiable foundation soil. Extensive data on the cyclic characteristics of the Nevada sand is reported by Arulmoli et al. (1992).

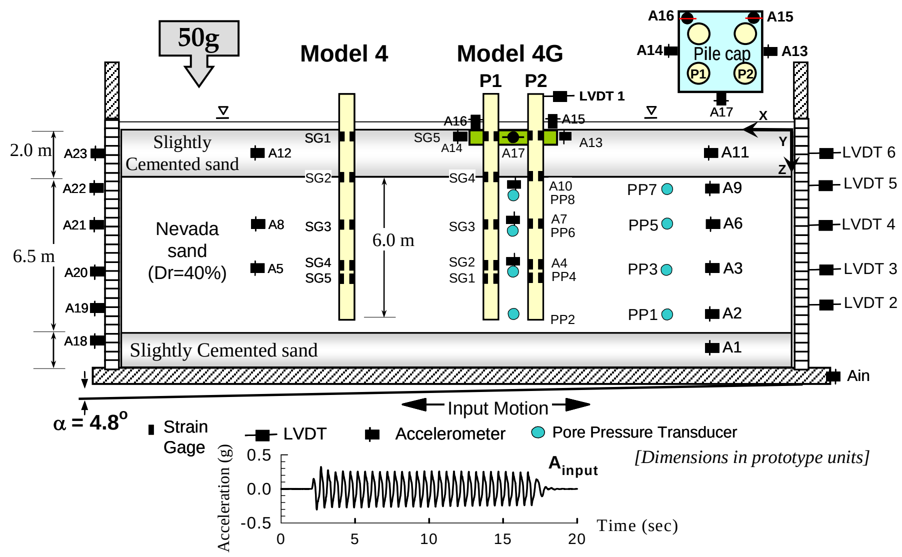

Figure 2. Test setups for floating single pile and group pile.

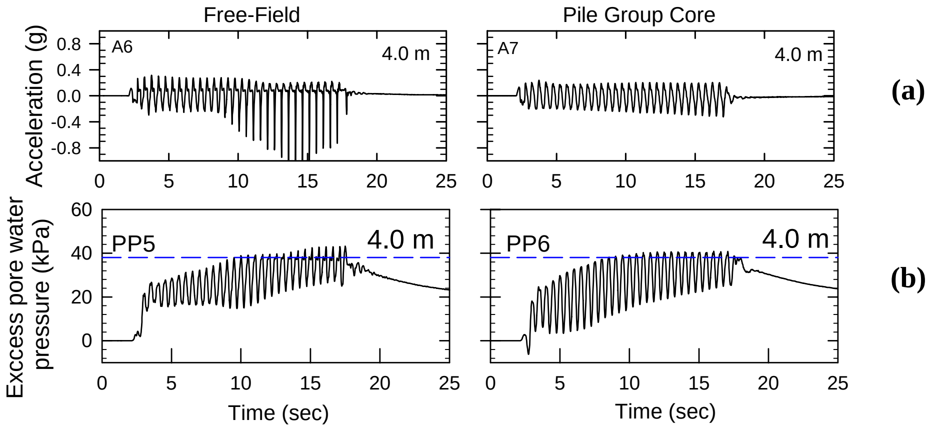

Figure 3. (a) Accelerations and (b) excess pore water pressures recorded in the free-field and pile group core, Model 4G.

The prototype profile includes a 1.5 m bottom layer of slightly cemented sand, topped by a 6.5 m layer of uniformly rained Nevada sand at a relative density of about 40%, topped by a 2.0 m layer of the same slightly cemented sand layer. Both of the cemented sand layers were initially utilized by Abdoun (1997) to simulate the stratified field geology that existed at the NFCH building site (Figure 1). The prototype pile group being simulated in both models, as shown in Figure 2, involves a 2×2 floating pile group, with a pile cap (W=2.9, L=2.9, H=0.66 m, in prototype units) embedded in the top slightly cemented sand layer. The piles in the pile group were spaced at 3d. Each of the piles had a diameter of about 0.60 m and EI of 8000 kN-m^2 (in prototype units). The locations of the strain gauges along the length of the model piles are also depicted in Figure 2. These gauges measured the bending moments in the piles imposed by the lateral spreading. The models were excited by a sinusoidal base motion (Figure 2) with uniform acceleration amplitude of about 0.25g and frequency of 2 Hz (in prototype units), parallel to the base of the inclined laminar box, at a centrifugal acceleration of 50g. Free-field response and pile cap-soil interaction were monitored by LVDTs, accelerometers and pore water pressure transducers.

TESTING RESULTS

In Model 4G, the acceleration records in the free-field in the loose Nevada sand layer (e.g., A3 in Figure 3a) showed significant drops in positive acceleration amplitudes while exhibiting a number of large spikes in the negative upslope direction after about several cycles of shaking, indicating an associated loss of soil stiffness and strength due to liquefaction. Since the liquefied soil cannot transmit shear stresses, the accelerometers (A11, A12, not shown here) in the top cemented sand also

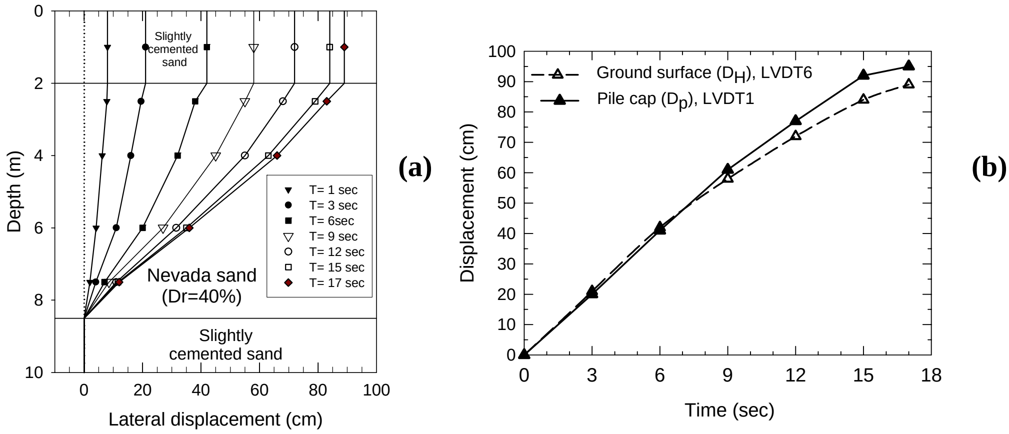

Figure 4. (a) Lateral displacement profiles in the free-field (LVDT 2~6); (b) free-field lateral ground surface deformation versus pile cap displacement, Model 4G.

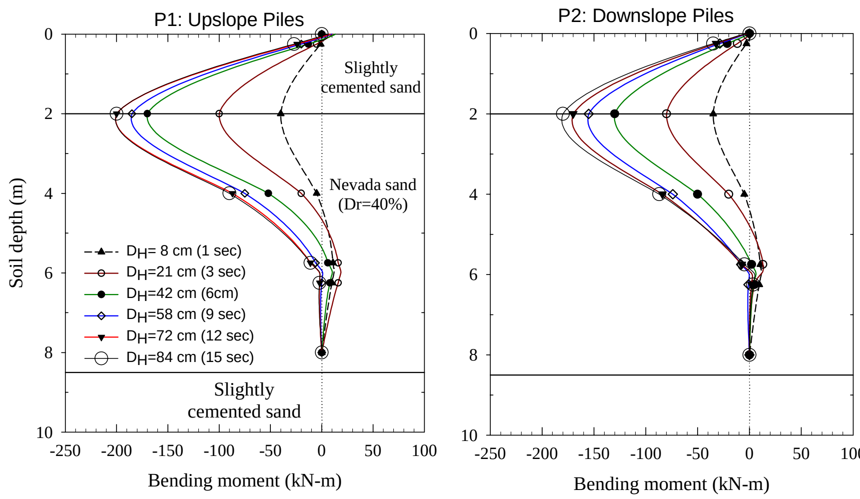

Figure 5. Bending moment profiles, upslope (P1) and downslope (P2) piles, Model 4G.

showed a significant drop in acceleration amplitude, indicating the top slightly cemented sand layer remained solid. These acceleration records, including the upslope spikes, were typical of those observed during lateral spreading in Model 4. The accelerations in the pile group core (e.g., A7 in Figure 3a) were less than those measured in the free-field, possibly due to the confining effects of the piles in the pile group core that prevented soil displacing or flowing in the downslope direction.

Figures 3b shows the excess pore water pressures recorded by pore water pressure transducers in both the free-field and the pile group core in Model 4G. Dashed lines in the figures show the initial vertical effective stresses at the elevations where the piezometers were located. All of the excess pore pressure measurements indicated the same behavior during and after shaking in both the free-field and the group pile core. That is, after the initial build-up of excess pore pressure, the pore pressure reached a plateau, and then pressure dissipation proceeded. The shapes of the pore pressure plateaus were very similar at all elevations, except the buildup pressures were slightly higher in the free-field than those measured in the pile group core (i.e., excess pore pressure ratio, r_u<1, in the pile group core), possibly due to confinement of the piles in the pile group core.

Figure 4a shows lateral displacement profiles at different times during shaking in Model 4G. These profiles, also referred to as displacement isochrones, were obtained from filtering out the cyclic component of the measured displacements. The final displacement profile at the end of shaking matched the manual measurements that were obtained after the test. At all times the maximum displacement (D_{Hmax}) occurred at the surface. The maximum permanent prototype displacement of the ground surface was about 90 cm at the end of shaking. This was consistent with Model 4 of Abdoun (1997). The measured permanent displacements at different times during shaking in the free-field (D_H) and at the pile cap (D_p) were compared in Figure 4b. Both of the displacements increased monotonically, and were practically the same for the first 9 sec of shaking. Thereafter, the pile cap

Table 1. Summary and comparison of results.

Test No.	Number of Piles	Moment M_{max} [kN.m]		Soil Profile
		Measured	Calculated(1)	
4(2)	1	120	99	2-layer
4G	4	180(3) ~ 200(4)	199	2-layer

(1):Based on limiting equilibrium analysis.
(2): *Abdoun (1997).*
(3)&(4): on upslope (P1) & downslope (P2) piles, respectively.

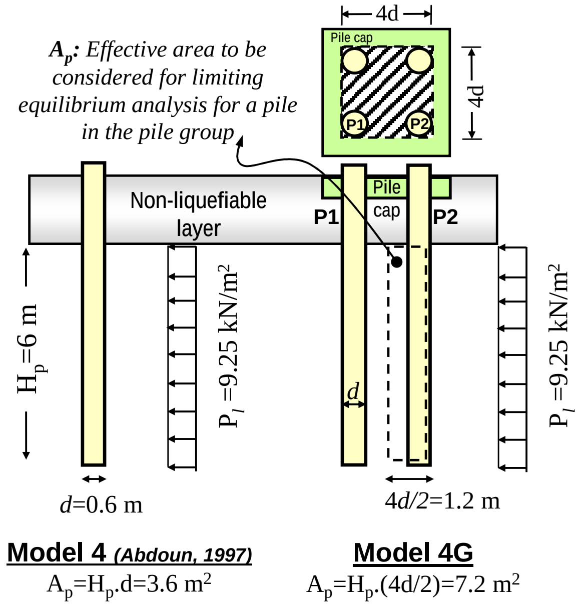

Figure 6. Free body diagram of the liquefied soil layer used for limiting equilibrium analyses of maximum pile bending moments of floating piles.

displacement (D_p) became slightly larger than the ground displacement (D_H). This was caused by failure of the cemented sand which caused a rotation of the pile cap in the shaking direction.

Figure 5 shows the moment profiles at different times (D_H=8 ~ 84 cm) during shaking along piles P1 and P2 in Model 4G. The maximum moments (M_{max}) were always measured at the upper boundary of the liquefiable Nevada sand layer and the values were 200 and 180 kN-m, respectively. The moments on P1 were slightly larger than that on P2 at all times during shaking (Table 1).

LIMITING EQUILIBRIUM ANAYSIS OF PILE GROUPS

In Model 4 (Abdoun, 1997), utilizing a floating single pile without a pile cap, the maximum moment was measured as about 120 kN at the upper interface. It was reported that the liquefied soil imposed a lateral pressure of about 9.25 kN/m2 on the single pile during lateral spreading. Limiting equilibrium analysis was proposed, assuming that the pile behaves like a cantilever beam. That is, the analysis assumed that the soil surrounding the single pile was fully liquefied, and then started flowing around it. The analysis was used for calculating the maximum bending moment at the interlayer. This limiting equilibrium was defined as;

$$M=A_p\, p_l\, H_p/2 \qquad (1)$$

Where:
A_p = Total pile area subjected to liquefied soil pressure
H_p = Pile length in liquefied soil
p_l = lateral soil pressure [9.25 kN/m2]

and predicted (calculated) the maximum moment as about 100 kN-m at the upper interface for a single floating pile (Table 1).

The comparison of measured bending moments for single and pile group models (not shown here) indicated that their behavior during liquefaction and lateral spreading were somewhat different. This difference may be caused by pile group effects in Model 4G. That is, the measured excess pore pressure buildups in the pile group core were slightly less than that in the free-field, indicating the pile core did not fully liquefy in Model 4G (Figure 3b). Similarly, the acceleration responses in the pile group core were slightly smaller than those in the free-field (Figure 3a), suggesting that the presence of the piles limited the lateral deformation of the soil enclosed by the pile group. Therefore, both the piles and the soil in the core area were penetrating into the liquefied soil together, or the liquefied soil in the free-field flowed around the pile group. In other words, the pile group behaves like a single pile as was observed in Model 4. In this case, the limiting equilibrium analysis shown in Equation 1 can not be used to predict the maximum moments on an individual pile in the pile group. Thus, a limiting equilibrium analysis was proposed for the pile group, as shown in Figure 6, based on the concept used in Equation 1, while utilizing the imposed lateral pressure from the liquefied soil (i.e., p_l=9.25 kN/m^2) obtained from Model 4. Accordingly, the total pile area subjected to liquefied soil pressure (A_p) in Equation 1 was modified so that pile group effects are taken into consideration, as shown below:

A_p = Total pile area subjected to liquefied soil pressure [$H_p \times (4d)/2 = 7.2$ m^2]

The maximum bending moments calculated based on A_p the defined above for the pile group was about 200 kN-m. The maximum bending moments measured on upslope (P1) and downslope (P2) piles are 200 and 180 kN-m, respectively. Both calculated and measured results are summarized in Table 1, indicating that they show good agreement.

CONCLUSIONS

Centrifuge tests were conducted to study the behaviors of floating single and pile groups. The results of tests on both floating single pile (Model 4) and pile group (Model 4G) foundations indicated that bending moments developed along the piles were controlled by the lateral pressures exerted by the liquefied layer. Measured moments were larger for pile group foundations due to the framing action (or group action) that limited lateral soil deformation caused by the piles in the pile group. A simple limit equilibrium procedure was proposed for pile groups based on the results from single pile model tests. The proposed analysis predicted the measured bending moments quite well, by using the concept of framing action limiting the lateral deformation in the pile groups.

REFERENCES

Hamada, M., Yasuda, S., Isoyama, R. and Emoto, K. "Study on liquefaction-Induced permanent ground displacements," Association for the Development of Earthquake Prediction in Japan, Tokyo, Japan, November 1, 123 p., 1986.

Hamada, M. and O'Rourke, TD. (eds.) "Case studies and liquefaction and lifeline performance during past earthquakes," Technical Report, NCEER-92-0001, Vol. 1, NCEER, Buffalo, NY, 1992.

Hamada, M. "Performance of foundations against liquefaction-induced permanent ground displacement," Proc. of the 12th WCEE, Jan. 30-Feb. 2, Auckland, New Zealand, Paper No. 1754, 8p., 2000.

Kawashima, K., Shimizu, K., Suzuki, N. and Nakamura, S. "Analytical studies on damages to bridges foundation piles caused by liquefaction-induced permanent ground displacement," Proc. 1st Japan-U.S. Workshop on Liquefaction, Large Ground Deformation and Their Effects on Lifeline Facilities, Tokyo, Japan, 1988.

Abdoun, T. "Modeling of seismically induced lateral spreading of multi-layer soil deposits and its effect on pile foundations," Ph.D. Thesis, Department of Civil Engineering, Rensselaer Polytechnic Institute, 1997.

Pamuk, A. "Physical modeling of retrofitted pile groups including passive site remediation against lateral spreading," Ph.D. Thesis, Department of Civil Engineering, Rensselaer Polytechnic Institute, 694p., 2004.

Arulmoli K. Muraleetharan KK. Hossain MM. and Fruth LS. "Verification of liquefaction analysis by centrifuge studies laboratory testing program soil data," Technical Report, The Earth Technology Corporation, Irvine, California: 404 p., 1992.