Effect of Liquefaction on Lateral Response of Piles by Centrifuge Model Tests

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This article presents work conducted on the effect of liquefaction on lateral pile response during the first year of NCEER's Highway Project. Research was conducted at Rensselaer Polytechnic Institute using the geotechnical centrifuge facility. For more information, contact Professor Ricardo Dobry, Rensselaer Polytechnic Institute, (518) 276-6934.

Many existing bridges are founded on piles driven through loose sand that may liquefy during earthquake shaking. Both lateral stiffness and lateral capacity of piles are very sensitive to the properties of the surrounding soil, be them friction or end-bearing piles. In current seismic analysis procedures, the effect of soil on lateral response is incorporated through nonlinear distributed soil springs along the pile within a beam-on-elastic foundation formulation. The pressure-deflection curves character-izing those springs, called p-y curves, depend on pile diameter, soil properties, and state of effective stresses (Cox, Reese and Grubbs, 1974; and Reese, Cox and Koop, 1974). Therefore, it is of great interest to evaluate the influence of the pore water pressure buildup in the sand due to the shaking on the p-y curves controlling the lateral response of the pile during the rest of the shaking. This is being done in this project by means of centrifuge model testing at the Rensselaer Polytechnic Institute 100 g-ton geotechnical centrifuge in Troy, New York. It is expected that this will result in a proposed guideline for seismic analysis of piles in liquefying sand.

Basic Model

The basic centrifuge model is shown in figure 1. An end-bearing model pile, with its tip fixed to the bottom of the box, is surrounded by saturated sand having a relative density, $D_r \cong 60\%$. Seismic shaking of limited duration is applied in-flight to the base of the rigid container to induce an excess pore pressure in the sand. At this stage, no relative displacement pile-soil is desired; the pile head is therefore kept locked and the pile moves together with the container during the shaking.

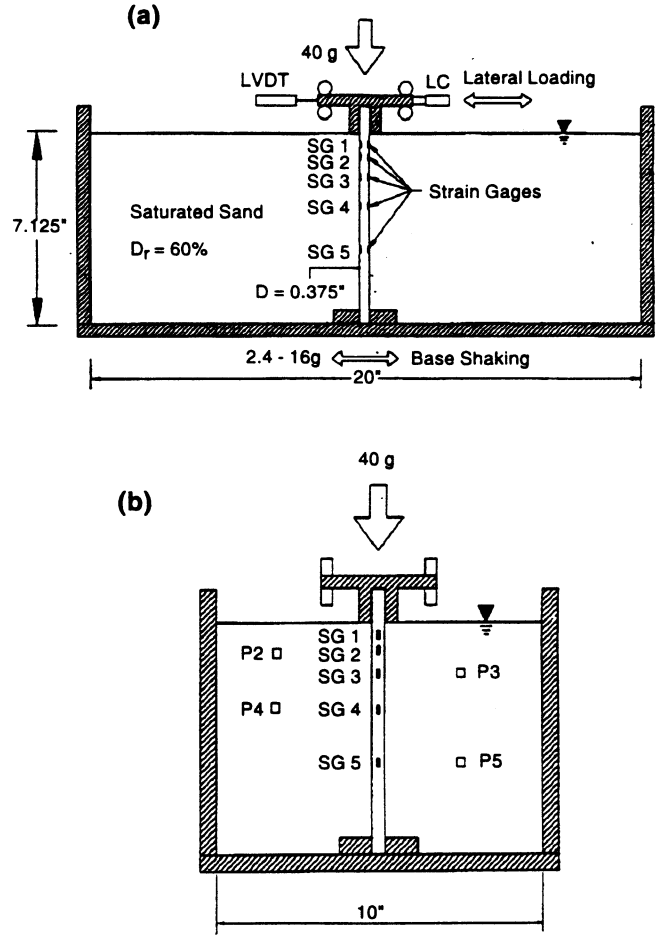

Figure 1: Centrifuge basic model and container: (a) side view; (b) front view.

Immediately after shaking, and while there are still excess pore pressures in the soil, the pile head is unlocked, and a cyclic (but static) lateral load test is conducted in-flight through a horizontal actuator located above the ground surface. During this load test, rotation of the pile is prevented, thus enforcing a fixed-head condition. The force-displacement relation at the pile head is measured with a load cell and with an LVDT, respectively; the bending strains along the pile are determined by means of strain gages (SG), and the excess pore pressures in the sand are monitored through miniature piezometers (P), as shown in figure 1. To avoid too rapid a dissipation of excess pore pressures after the end of shaking due to the increased permeability of the soil in the high g-field, a deaired water-glycerol mixture is used as pore fluid, which has a viscosity about 10 times greater than water.

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The purpose of this centrifuge model is to establish the effect of excess pore pressure in the sand on the p-y curves at different depths along the pile. Most of the tests in the project use the basic model just described, and do not involve structural inertia forces. A test involving a mass on top of the pile during base shaking, so as to develop truly seismic loading rather than cyclic loading through an actuator, is planned for a later date for verification purposes.

A prototype steel pipe pile 22 ft. long, 15 in. outside diameter, and with a bending stiffness EI = 9.95×10^6 kip-in^2 was selected as reasonably representative of many highway bridge foundations. After taking into account the scaling factor of 40 for all linear dimensions for a 40-g centrifugal field, a model brass pile with the properties listed in table 1 was selected.

Table 1: Model Pile Properties

Length (in)	O.D. (in)	I.D. (in)	Material	Modulus of elasticity (psi)
6.625	0.375	0.347	Brass	15×10^6

Model Preparation and Test Procedure

The soil deposit has a dimension of 20 in. (L) x 10 in. (W) x 6.625 in. (H), simulating a prototype scale saturated sand deposit of about 22 feet thick resting on stiff bedrock. The model pile is installed in the model container with its tip fixed at the bottom. Dry Nevada No. 120 sand is then drained into the container with relative densities in the range of 62 ± 3%. Pore pressure transducers are installed at various depths during this process. Compaction around the pile is applied by layers to minimize the difference between the actual driving process and the installation process used in the test.

The soil model is then vacuumed and saturated with the deaired water-glycerol mixture. The resulting permeability of the prototype soil deposit being modeled is 10^{-2} cm/s. After saturation, the loading unit is installed on the model, the computer-operated actuator locks the pile head electronically in a neutral position and a zero slope boundary condition at the pile head during the test is secured. The pile-soil model is then spun up to 40 g in the centrifuge for consolidation. Base shaking is applied at the model base in flight after the soil stratum is fully consolidated and all instruments have reached steady state. The shaking and lateral loading are synchronized in such a way that immediately after base shaking ends, the computer unlocks the actuator and the lateral loading at the pile head starts. In practice, this means that the lateral loading starts 100 milliseconds (4 seconds in prototype time) after the start of base shaking. Data from 16 channels are acquired at 50 kHz and saved directly on the computer hard drive.

Testing Program

The main centrifuge model testing program is shown in table 2. These tests have all been completed. Test PL16 was conducted without soil, and Test PS01 with soil but no shaking. The rest of the tests included both shaking followed by a cyclic lateral load test at the pile head, as already described. The average base acceleration applied to the system during the shaking stage, as listed in the table, is in prototype units; that is, actual horizontal accelerations 40 times larger were applied in-flight to the base of the model. The values of r_u listed give the range of maximum excess pore pressure ratios measured by the piezometers at various depths.

Table 2: Model Testing Program

Tests	Soil	Average Base Shaking Acceleration (g)	Range of Maximum r_u Over Deposit Thickness
PL16	No	—	—
PS01	Saturated	No	—
PS02	Saturated	0.400	100%
PS03	Saturated	0.145	61% - 100%
PS06	Saturated	0.060	32% - 100%
PS07	Saturated	0.340	95% - 100%

Test Results and Preliminary Interpretations

The model pile was first calibrated in Test PL16 while spinning the centrifuge at 40 g without placement of any soil in the model container. No shaking was done in this test. Lateral loading was applied at the pile head while in-flight. The pile stiffness, boundary conditions, pile head displacement, and force and bending moments

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were verified with the theoretical solutions for a pile without soil fixed at both ends, with good agreement.

The pile-saturated soil model was then calibrated in Test PS01 by lateral loading in flight without any base shaking. A set of p-y curves was obtained from the measurements, following the same method typically used to develop conventional p-y curves from full scale pile loading tests in the field. These p-y curves obtained from Test PS01 are summarized in figure 2. Figure 3 compares the measured bending moments along the pile (data points) with those predicted using these p-y curves (lines) for several values of the pile head displacement. The figure also includes comparisons of predicted and measured pile head force F_0.

Next, Tests PS02 to PS07, all of which involving shaking followed by lateral loading, were conducted to observe the p-y response at various levels of pore pressure ratio in the sand. The only difference between these various tests was the amplitude of base shaking acceleration, which in turn developed different levels of pore pressure ratio (table 2). Selected short term and long term records measured in Test PS07 are plotted in figures 4 and 5 in prototype units.[1] Figure 4 includes the following measured time histories: (a) base horizontal acceleration, (b) pore pressure ratio at a depth of 8.7 feet, (d) pile head lateral displacement, (f) pile head force, and (c) and (e) two of the pile bending moments measured by the corresponding strain gages. The average amplitude of the input base acceleration in this test was 0.34 g, strong enough to liquefy the soil stratum almost completely. It can be seen that the pore pressure ratio r_u reached 100% very rapidly, as shown in figure 4(b). The pile head was locked during the shaking (no displacement); still, a cyclic lateral force and cyclic bending moments along the pile were measured during shaking due to inertial forces developed in the loading unit and the soil. Figure 5 shows the long term time histories of: (a) pile head lateral displacement y_0,

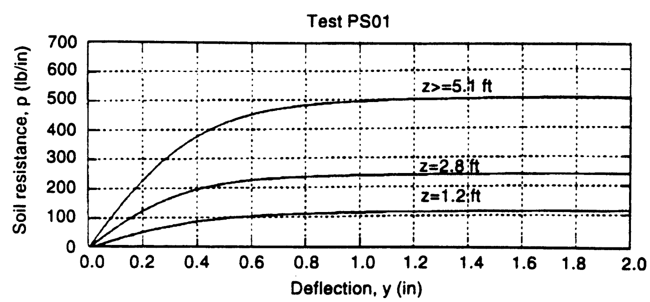

Figure 2: p-y curves calibrated in test PS01.

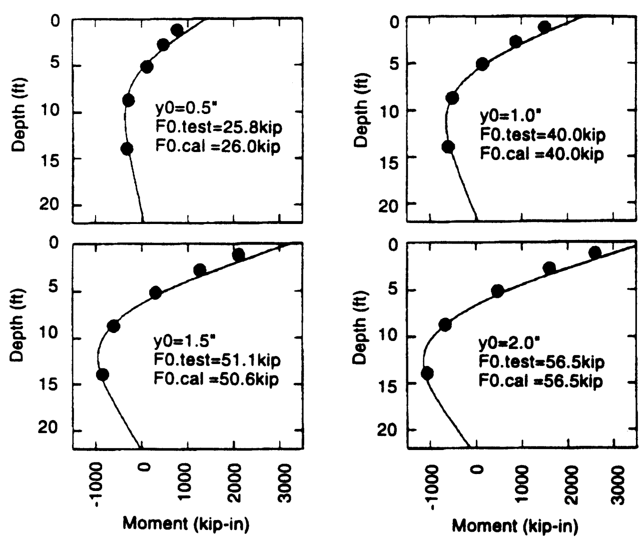

Figure 3: Measured and calculated bending moments for lateral load test in test PS01.

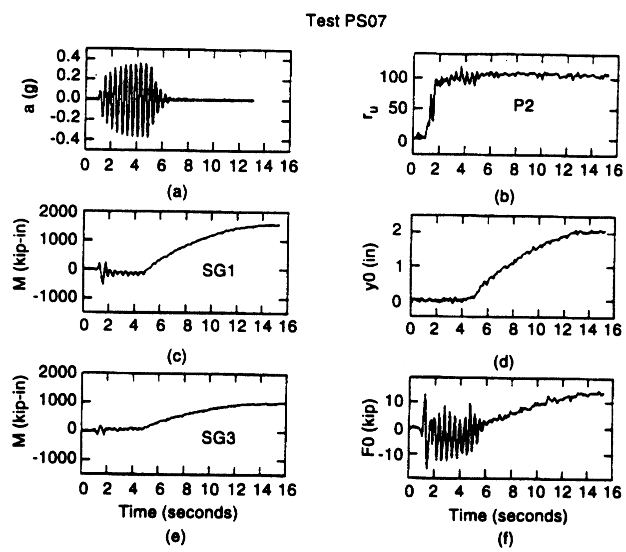

Figure 4: Short term time histories in Test PS07: (a) base acceleration, (b) pore pressure ratio at a depth of 8.7 ft., (d) pile head lateral displacement, (f) force at pile head, and bending moments measured with strain gages at (c) SG1 and (e) SG3.

[1] To get these prototype units, the actual model measurements have been multiplied by a scaling factor as follows: a factor of 40 for time and displacement y_0, a factor of $(40)^2 = 1{,}600$ for force F_0, a factor of 1/40 for acceleration a, and of $(40)^3$ for moment M; the pore pressure ratio r_u has a scaling factor of unity.

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(b) force F_0, and (c) pore pressure ratios r_u at various depths, during and after shaking. The small gap in the records at about 53 seconds was caused by an unexpected interruption of the data acquisition system, when the acquisition rate was switched from fast to slow. Fortunately, the gap is small and the missing data can be easily interpolated. As observed previously, at any given time the pore pressure ratio was not constant with depth; instead, it was usually greater at shallow elevations.

Figure 5 shows some of the key data provided by the lateral load test conducted after the end of the shaking (t > 5 seconds). A slowly varying lateral cyclic displacement of ± 2 inches was applied to the head of the pile. The frequency of the loading was low enough so that it induced no significant inertia forces. The corresponding force-displacement relation could be correlated with the pore pressure ratio simultaneously measured in the soil (figure 5(c)). As the pore pressures

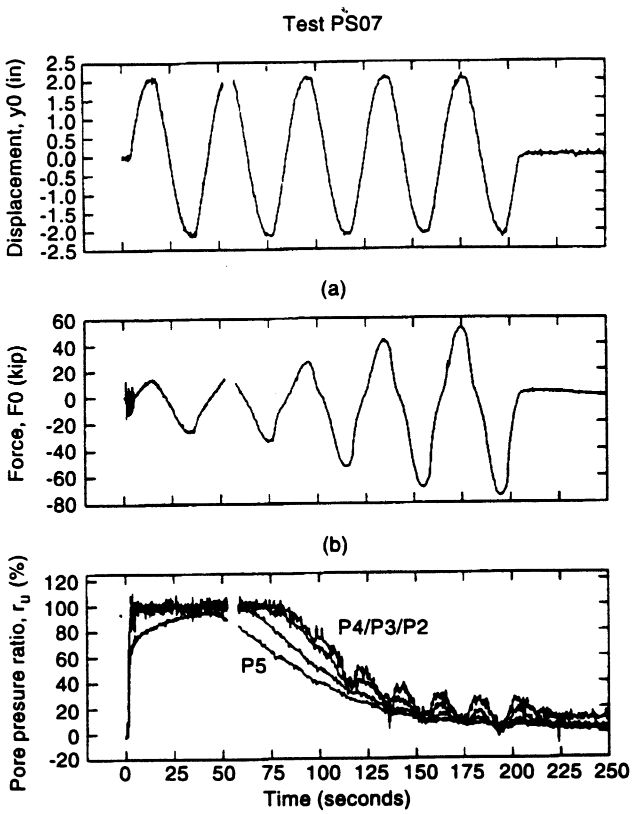

Figure 5: Long term time histories in Test PS07: (a) pile head lateral displacement, (b) force at pile head, and (c) pore pressure ratios at various depths.

dissipated with time, the soil stiffened and the force needed to reach the 2 inch displacement increased (figure 5(b)), thus providing in one test measurements ranging all the way from $r_u = 100\%$ to $r_u = 0$.

Measurements of pore pressures such as those illustrated in figure 5(c) showed cyclic fluctuations during the application of cyclic load at the pile head, especially for shallow depths and when r_u was low. This suggests that dilation occurred due to the pile deflection, as the pore pressure transducers were installed only about 3.5 ft. from the pile. The pore pressures measured by the piezometers were used directly in the analysis, with no attempt to separate the pore pressure into components caused by prior shaking and by pile deflection.

Correlation of p-y Curves With Pore Pressure Ratio

The lateral force F_0 measured at the pile head when y_0 = ± 2 inches, is plotted in figure 6 versus the pore pressure ratios measured in the soil at the same time. Figure 6 includes data from Tests PS02 to PS07, and from all relevant loading cycles. Each value of force is related to a range of pore pressure ratios at various depths, as defined by the corresponding bar in figure 6. In most cases, the right end of the bar is associated with pore pressure ratios at shallow depths, while the left end corresponds to deep elevations. The lateral force at a 2 inch displacement in Test PL16, without soil, and that in Test PS01, with soil but without shaking, have been plotted as data points in figure 6. These two data points bound all possible values of the pile head force: maximum possible force (soil and zero pore pressure ratio), and minimum possible force (no soil). The measurement bars in figure 6 fall between these two bounds, with the value of lateral forces decreasing as the pore pressure ratio increases, more or less following a linear pattern.

A more precise, but still preliminary, analysis of the data contained in figure 6 was conducted, using program LPILE and an assumed law relating pore pressure ratio and degradation of the p-y curves for $r_u = 0$ determined in figure 2. In this way, the large scatter of the measurement bars of r_u in figure 6 was significantly reduced, as shown in figure 7. In this plot, dimensionless degradation parameter C_u is more or less uniquely correlated with r_u.

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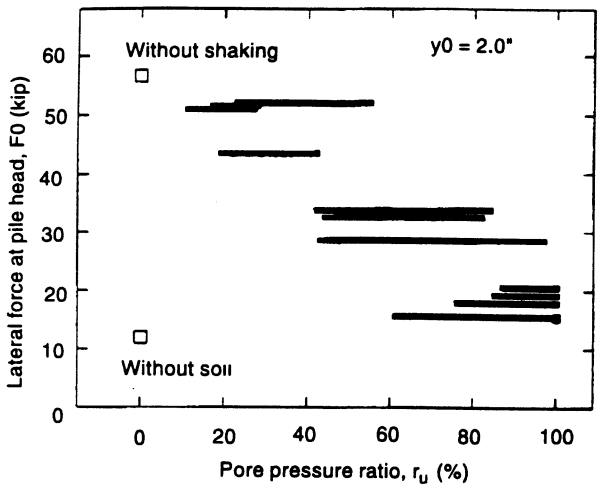

Figure 6: Lateral force at pile head versus pore pressure ratios for a 2 inch pile head lateral displacement for all tests.

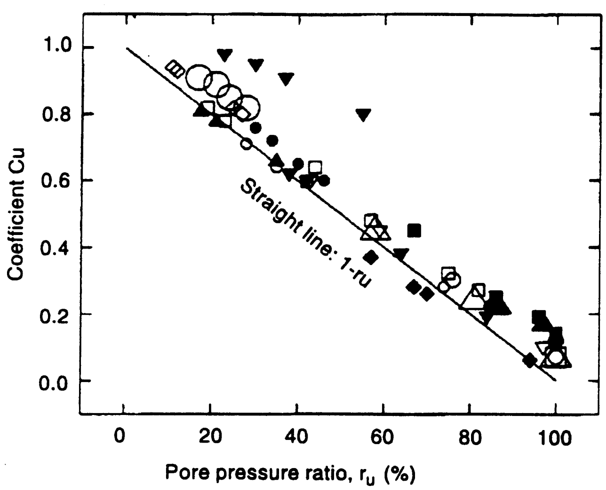

Figure 7: Pore Pressure Ratio r_u versus Degradation Parameter C_u.

Figure 8 shows the results of using the new, degraded p-y curves, including C_u obtained from figure 7, in the prediction of results measured in Test PS07. The measured pore pressure ratio distributions with depth are shown at the right hand side, while the predicted pile head lateral force and bending moments are included in the left-hand side of figure 8. The predicted bending moment lines compare very well with the data points measured with the strain gages. Comparisons such as figure 8 and other analyses will be used to support the proposed guidelines for the development of degraded p-y curves in a soil totally or partially liquefied by earthquake shaking.

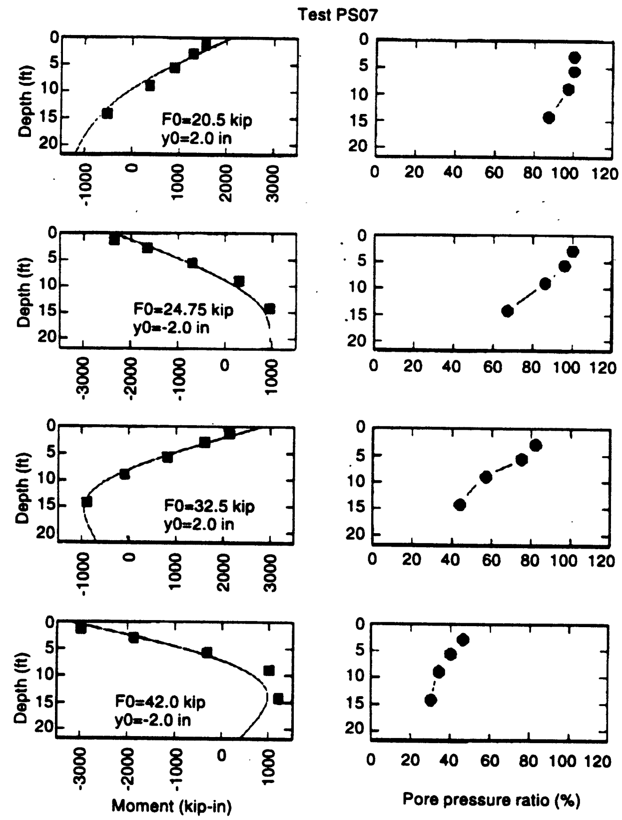

Figure 8: Selected Measured (Squares) and Calculated (Lines) Pile Bending Moments and Associated Pore Pressure Distributions in Test PS07.

References

Cox, William R., Reese, Lymon C., and Grubbs, Berry R., (1974), "Field Testing of Laterally Loaded Piles in Sand," Sixth Annual Offshore Technology Conference, Houston, Texas.

Reese, Lymon C., Cox, William R., and Koop, Francis D., (1974), "Analysis of Laterally Loaded Piles in Sand," Sixth Annual Offshore Technology Conference, Houston, Texas.

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