# Centrifuge-Based Evaluation of Pile Foundation Response to Lateral Spreading and Mitigation Strategies

*by Ricardo Dobry, Tarek H. Abdoun and Thomas D. O'Rourke*

## Research Objectives

A first objective of the research is to identify and quantify the mechanisms and parameters determining the hazard to deep foundations and superstructure caused by the lateral spreading. Two main situations have been identified, depending on pile foundation bending response controlled by the pressure of the liquefied soil or by that of a shallow nonliquefied layer. Field evidence, centrifuge results, and analyses have shown the shallow nonliquefiable layer to be more critical and more amenable to retrofitting. A second objective of the research is to develop and evaluate strategies to retrofit deep foundations by decreasing the pressure exerted by this nonliquefied shallow layer, with emphasis in cost-effective and advanced materials. A third objective is to develop fragility curves for nonretrofitted and retrofitted foundations.

The effects of liquefaction on deep foundations are very damaging and costly. Permanent lateral ground deformation or lateral spreading is a main source of distress to piles, either alone or in combination with inertial superstructural forces and moments arising during shaking and acting on a soil already weakened by rising water pore pressures. Cracking and rupture of piles at shallow and deep elevations, rupture of pile connections, and permanent lateral and vertical movements and rotations of pile heads and pile caps with corresponding effects on the superstructure have been observed (Figure 1). This has affected buildings, bridges, port facilities and other structures in Japan, the U.S. and other countries including the 1989 Loma Prieta, California and the 1995 Kobe, Japan, earthquakes (Hamada and O'Rourke, 1992; O'Rourke and Hamada, 1992; Tokimatsu et al., 1996; Dobry and Abdoun, 2001).

Examination and analysis of case histories have revealed important aspects of the phenomenon and highlighted its complexity. It is essentially a kinematic soil-structure interaction process involving large ground and foundation permanent deformations, with the deep foundation and superstructure responding pseudostatically to the lateral permanent displacement of the ground.

While in some cases the top of the foundation displaces laterally a distance similar to that in the free field, like in Figure 1 where both the

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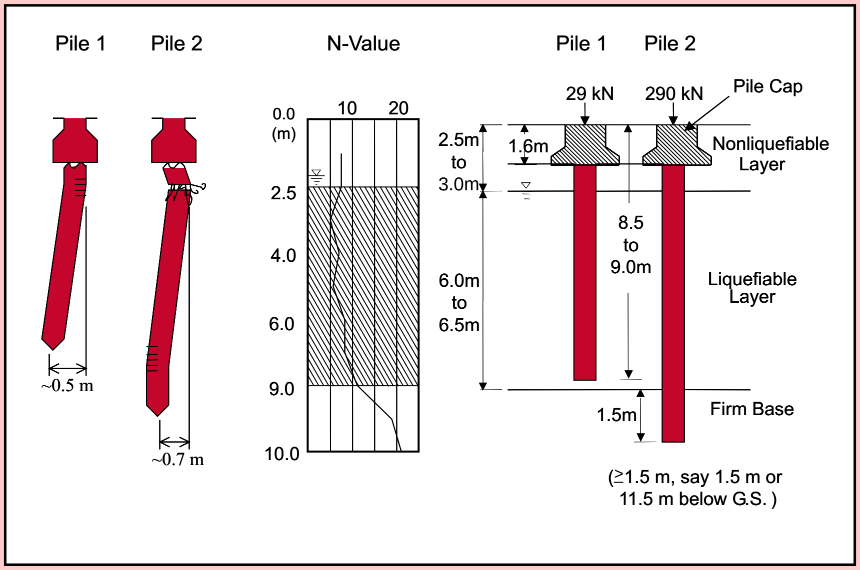
ground and foundation moved horizontally about 1 m, in others it moves much less due to the constraining effect of the superstructure, or of the deep foundation's lateral stiffness including pile groups and batter piles. The foundation may be exposed to large lateral soil pressures, including especially 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); while in others the foundation has failed first in bending (Figure 1) 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 (Figure 1). More damage tends to occur to piles when the lateral movement is forced by a strong nonliquefied shallow soil layer (end-bearing pile No. 2 in Figure 1), than when the foundation is freer to move laterally and the forces acting on them are limited by the strength of the liquefied soil (floating Pile No. 1 in Figure 1).

Lateral spreading has been identified as a major hazard to pile foundations of hospital buildings, and centrifuge modeling as a key tool to identify and quantify mechanisms, calibrate analyses and evaluate retrofitting strategies for pile foundations. Figure 2 shows the 100 g-ton RPI geotechnical centrifuge used for this research, which is located at the RPI campus in Troy, New York. This centrifuge, originally commissioned in 1989 with support from MCEER (then NCEER), has in-flight earthquake simulation capability allowing base shaking to be applied to the base of the model. It was recently selected by NSF together with other earthquake en-

**Engineering design firms, foundation and consulting engineers, hospital authorities, state transportation departments, and port and harbor authorities will all be interested in the results obtained through this research. The work is part of a broader geotechnical task focusing on ways to remediate potentially dangerous sites and/or rehabilitate deep foundations of hospitals and other structures vulnerable to earthquakes, by reducing or eliminating the effects of soil liquefaction, including those due to permanent lateral and vertical ground deformations. Lateral spreading has been identified as a major hazard to deep foundations, and centrifuge modeling is a key tool to identify and quantify mechanisms, calibrate analyses and evaluate retrofitting strategies for pile foundations. This centrifuge-based research provides the foundation component to the effort to mitigate the seismic hazard to hospitals with the help of cost-effective and advanced materials and technologies.**




■ Figure 1. Damage to pile foundations due to lateral spreading under NFCH building, 1964 Niigata earthquake, Japan (Hamada, 1992, 2001)

**Links to Current Research**

*Program 2: Seismic Retrofit of Hospitals*

gineering experimental sites throughout the U.S. to form the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES, *www.eng.nsf.gov/nees*). Additional information on the centrifuge equipment used in this research, results from other projects and the basic principles of centrifuge modeling, can be found at the RPI web site (*www.ce.rpi.edu/centrifuge*), which also has useful links to other relevant web sites; see also summary articles by Dobry et al. (1995) and Dobry and Abdoun (1998, 2001). In addition to the centrifuge experiments themselves done at RPI, this centrifuge-based research has included other analytical, laboratory, case history review and retrofitting strategy components, conducted either at Cornell University or in close cooperation between the RPI and Cornell teams. The RPI-Cornell joint centrifuge-based research on lateral spreading effects on piles started in 1995 with support from NCEER and NSF and has continued since then with current support from both MCEER and NSF. The technical discussion below is divided in three parts: case of pile bending response to lateral spreading controlled by the pressure of the liquefied soil, case of response controlled by shallow nonliquefied soil layer, and pile retrofitting strategies and results.

■ Figure 2. 100 g-ton geotechnical centrifuge with in-flight shaking capability at RPI

**Web Sites**

***RPI 100 g-ton geotechnical centrifuge facility:***
*http://www.ce.rpi.edu/centrifuge*
*The complete visualization or "movie" produced from the data recorded in the centrifuge test illustrated by the single frame of Figure 5 can be viewed in this web site. To see this movie, after entering the web site, go to Research and then to Visualization.*

***NEES Initiative:***
*http://www.eng.nsf.gov/nees*


# Pile Bending Response Controlled by the Liquefied Soil

Figure 3 shows centrifuge pile Model 3, simulating the bending response of a pile foundation subjected to the lateral pressure of a liquefied soil due to lateral spreading. These and other experiments were conducted using the rectangular, flexible-wall laminar box container sketched in Figure 3. This laminar box is comprised of a stack of up to 39 rectangular aluminum rings separated by linear roller bearings, arranged to permit relative movement between rings with minimal friction. In Model 3 as well as in all other lateral spreading spreading experiments, the laminar box and the shaker under it are inclined a few degrees to the prototype horizontal direction to simulate an infinite mild slope and provide the shear stress bias needed for a lateral spread. The flexibility of this box container is demonstrated by the large permanent deformations and strains attained in the experiments (Figure 5).

In the test of Figure 3, the soil profile consists of two layers of fine Nevada sand saturated with water: a top liquefiable layer of relative density, $D_r$ = 40% and 6 m protoype thickness, and a bottom slightly cemented nonliquefiable sand layer having a thickness of 2 m. The prototype single pile is 0.6 m in diameter, 8 m in length, has a bending stiffness, EI = 8000 kN-m$^2$, and is free at the top. The pile model is instrumented with strain gages to measure bending moments along its length, and a lateral LVDT at the top to measure the pile head displacement. The soil is instrumented with pore pressure transducers (piezometers) and accelerometers, as well as with lateral LVDTs mounted on the rings of the flexible wall to measure soil deformations in the free field. A prototype input accelerogram consisting of 40 sinusoidal cycles of a peak acceleration of 0.3 g was applied to the base, which liquefied the whole top layer in a couple of cycles and induced a permanent lateral ground surface displacement in the free field of about 0.8 m.

Results of this experiment are shown in Figures 4 and 5. As soon as the top sand layer liquefied at the beginning of shaking, it started moving laterally downslope throughout the shaking, with the maximum displacement at all times measured at the ground surface, and with this surface ground displacement increasing monotonically with time to its final value $D_H$ = 0.8 m at the end of shaking. The maximum bending moment along the pile at any given time occurred at the interface between the two soil layers, that is at a depth of about

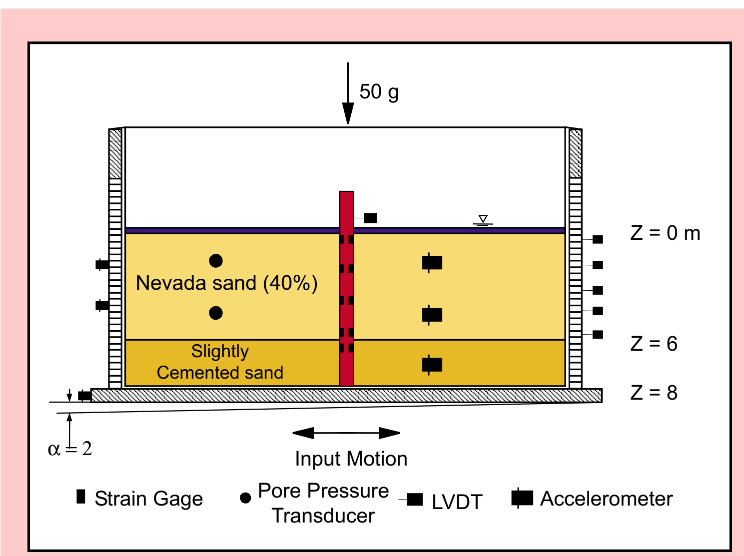
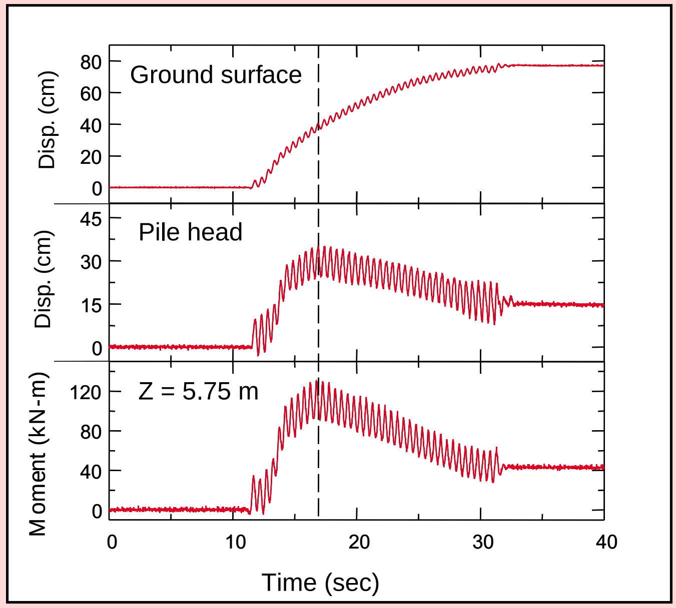


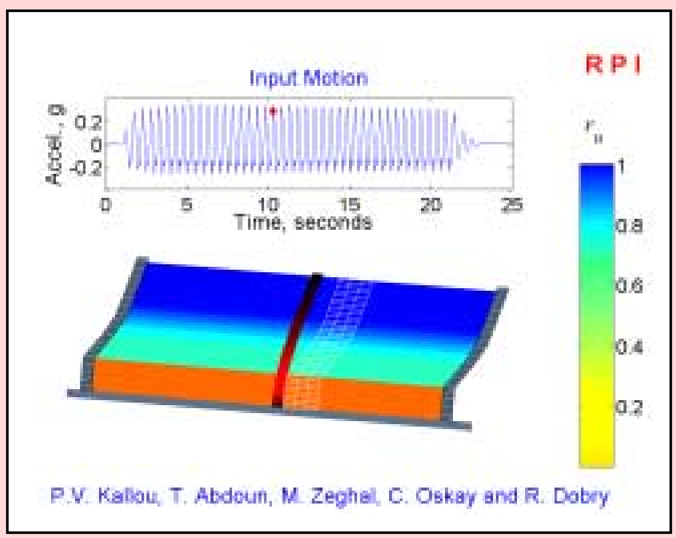

Figure 3. Lateral spreading pile centrifuge model in two-layer soil profile (Abdoun, 1997)

6 m. Figure 4 shows the time history of this prototype bending moment for Model 3, measured at z = 5.75 m; the plot reveals that the moment increased to a maximum $M_{max}$ = 110 kN-m at a time, $t^a \approx 17$ sec, with the moment decreasing afterwards despite the continuation of shaking and the continuous increase of the soil deformation in the free field. The pile head displacement in the same figure also reached a maximum at about 17 sec and decreased afterwards. Clearly at this time the liquefied soil reached its maximum strength and applied a maximum lateral pressure to the pile, with the soil flowing around the pile, exhibiting a smaller strength and applying a smaller pressure afterwards; as a result, the model pile bounced back and the bending moments decreased. The two photos in Figure 6 - taken after the centrifuge tests - illustrate this flow of liquefied soil around the pile in other two models where colored sand had been placed around the pile.

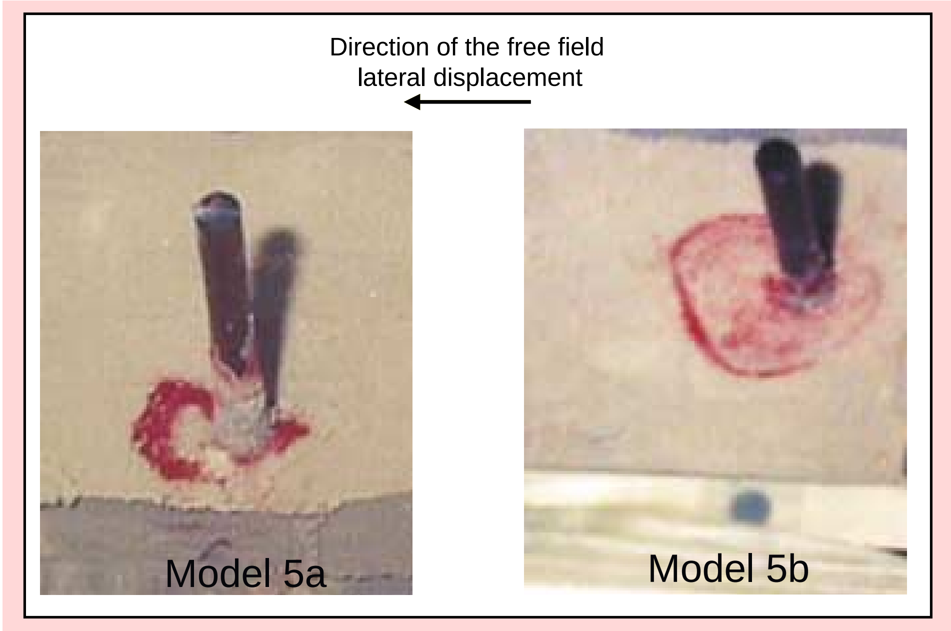
Figure 5 summarizes the state of the system during a repeat of Model 3, at the time when the pile head displacement and the bending moment at a depth of about 6 m attained their maximum values. This is a frame taken from the visualization of the experiment produced from the measurements (the whole visualization may be viewed at the RPI centrifuge web site). The displaced shape of the box container indicates the lateral spreading in progress, with concentration of permanent shear straining in the lower part of the liquefied soil; this box shape was obtained from the lateral LVDTs placed on the side walls. This distorted shape is also copied as a white mesh to the right side of the pile for direct comparison between ground and pile displacements as well as to visualize the larger movement of the liquefied




■ Figure 4. Prototype lateral displacement of soil and pile and ground surface, and pile bending moment at a depth of 5.75 m in model of Figure 3 (Abdoun, 1997)



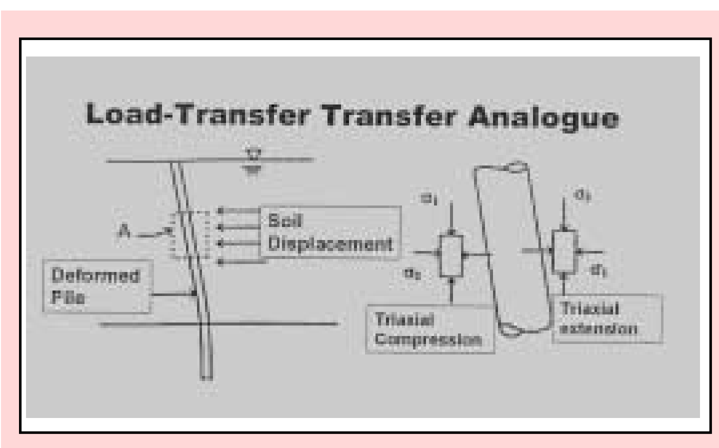

■ Figure 5. Frame taken out of visualization of two-layer centrifuge model of Figure 3, produced from the recorded data (Kallou et al., 2001; to see whole visualization, visit *http://www.ce.rpi.edu/centrifuge*)



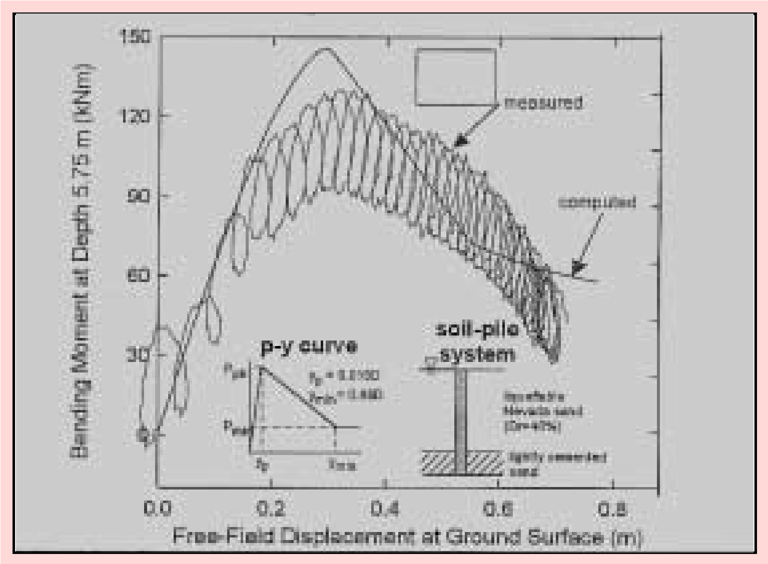

■ Figure 6. Photos showing flow of liquefied sand around the pile in the downslope direction in two-layer centrifuge models (Abdoun, 1997). The photos were taken after the test in models where colored sand had been placed in a circular ring around the pile

soil flowing around the pile, compared with the displacement of the pile itself. The blue color in the upper part of the loose sand layer indicates complete liquefaction as measured by the piezometers, while the green color in the lower part of the layer indicates lower excess pore pressure due to dilative cyclic stress-strain response of the liquefied sand in that part of the shaking cycle. At other times corresponding to different parts of the shaking cycle, the whole layer is blue and hence completely liquefied.

In addition to Model 3 summarized in Figures 3 to 5, similar centrifuge tests of a single pile with a pile cap, with densification around the pile to simulate pile driving, and with 2x2 pile groups indicated that, while $M_{max}$ still occurs at a depth of about 6 m sometime during the shaking, the value of $M_{max}$ increases with the area of pile foundation exposed to the soil lateral pressure and decreases in the pile groups due to the contribution to moment of the axial forces in the piles (frame effect). Simple limit equilibrium calculations with a constant assumed maximum pressure of the liquefied soil along the pile, $p_l$, indicate that values of $p_l$ of the order of 10 kPa explain well all measured trends and values of $M_{max}$ in this series of centrifuge tests.




■ Figure 7. Concept used to develop undrained triaxial extension model for the lateral loading of liquefied soil on the pile (Goh and O'Rourke, 1999; Goh, 2001)

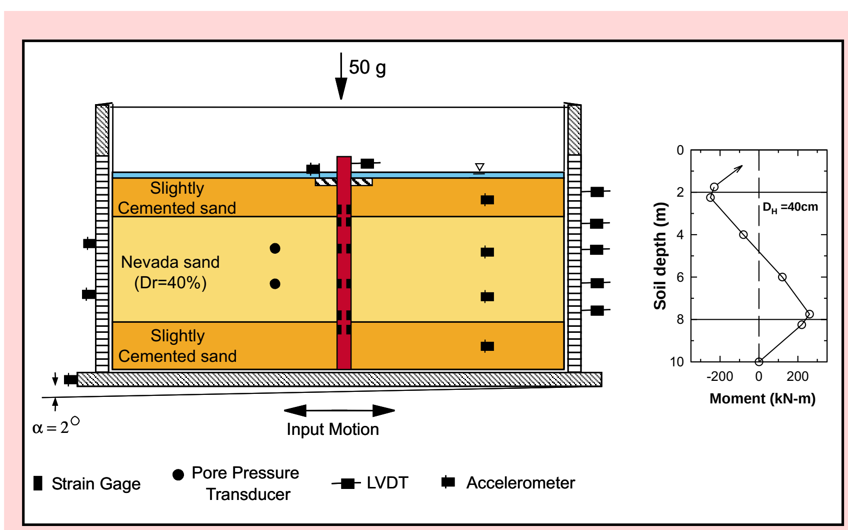



■ Figure 8. Comparison between predicted and measured pile bending moment in centrifuge model of Fig. 3 at the lower boundary of liquefied soil using triaxial extension undrained loading approach (Goh and O'Rourke, 1999; Goh, 2001)

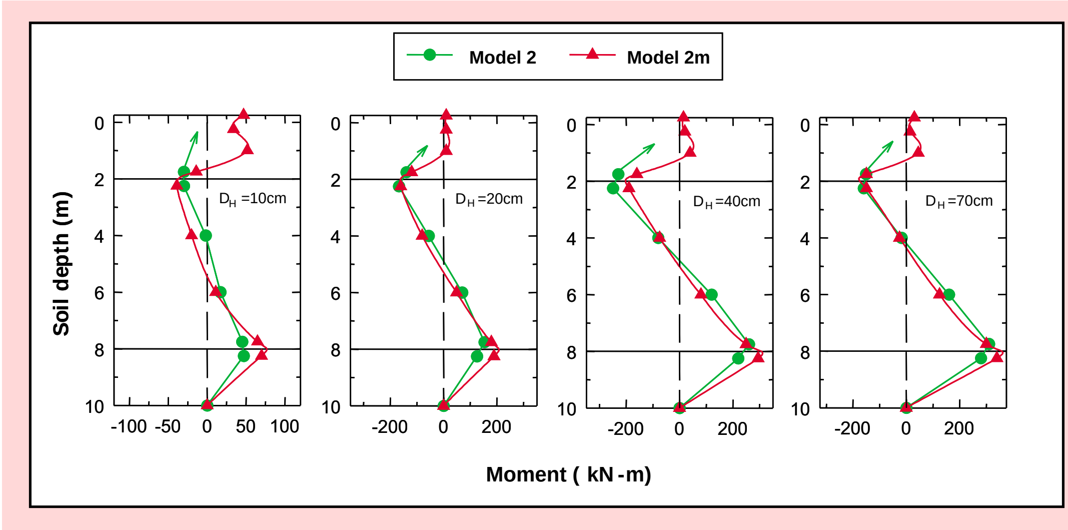
The physical origin and basic mechanisms determining the behavior of the liquefied soil, including the lateral pressure on pile foundations and values such as $p_l$ and $M_{max}$ measured in these centrifuge tests, are not yet well understood and are the subject of intense research. The Cornell team has proposed the explanation sketched in Figure 7, with $p_l$ and $M_{max}$ controlled by the peak undrained shear strength of the saturated sand loaded in the extension mode (Goh and O'Rourke 1999; Goh, 2001). Based on p-y curves generated analytically from triaxial extension tests conducted at Cornell using the same Nevada sand and relative density of the centrifuge tests, nonlinear Beam-on-Winkler-Foundation (BWF) analyses of centrifuge Model 3 were able to predict closely the measured bending response (Figure 8).

## Pile Bending Response Controlled by Shallow Nonliquefied Layer

Figure 9 shows centrifuge Model 2, where a strong shallow nonliquefied soil layer increases significantly the bending response of the pile foundation to lateral spreading. The shallow top layer




■ Figure 9. Lateral spreading pile centrifuge model in three-layer soil profile (Abdoun, 1997)




■ Figure 10. Measured bending moment response along pile in lateral spreading centrifuge models without (Model 2) and with (Model 2m) inertial loading (Wang, 2001)

consists of a 2-m thick (in prototype units), free draining, slightly cemented sand. Model 2m, not shown here, is similar to Figure 9 but with a mass added above ground to evaluate the combined effects of lateral spreading and inertial loading.

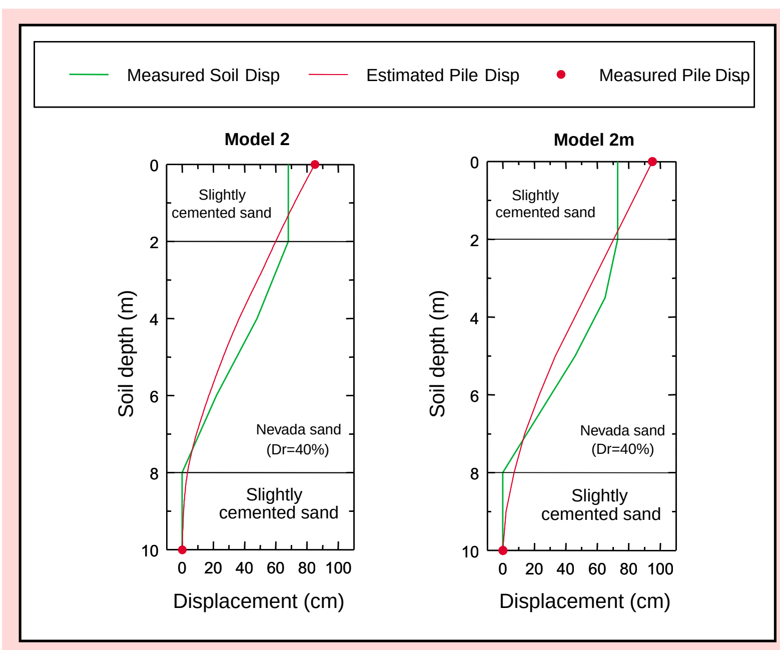
Figures 10-11 summarize the main characteristics of the bending response of Model 2 (only lateral spreading), which is also typical of other pile models tested in this 3-layer soil profile. The same as in Model 3 discussed before, the 6-m thick noncemented sand layer liquefied early in the shaking after which the lateral spreading increased monotonically, reaching a value $D_H$ = 0.7 m at the end of shaking (Figure 11). The pile bending moments in the top 2 m first increased with time of shaking and then decreased after passive failure of the top nonliquefiable layer against the pile (Figure 10); while the bending moments near the bottom increased monotonically and never decreased, as the bottom nonliquefiable layer did not fail. The values of maximum bending moments at 2 m and 8 m are close to 300 kN-m, much greater than those measured in 2-layer tests such as shown in Figure 3, which did not exceed 170 kN-m even when a pile cap was added.

The shapes of the bending moment profiles at various times presented in Figure 10 indicate that the deformed shape of the pile had a double curvature caused by the top and bottom soil layers loading the pile in opposite directions. This double curvature was confirmed by the fact that when the top soil layer failed, the pile head and cap "snapped" in the downslope direction (Figure 11), showing that at very shallow depths, the pile was pushing the soil rather than the other way around. Both the passive failure of the top layer and the moment concentrations at the top and bottom boundaries of the liquefied layer indicated by the figures are consistent with the experience from earthquake case histories.

These moment concentrations are also predicted by theory (e.g., Meyersohn, 1994; Meyersohn et al., 1992; Debanik, 1997). Another interesting aspect of Figures 10 and 11 is that the bending moments vary linearly within the liquefied layer, suggesting that they are essentially controlled by the loading of the top and bottom layers, with the pressure of the liquefied soil being negligible. The values of $M_{max}$ at z = 2 m and z = 8 m are higher than the corresponding values of $M_{max}$ at z = 6 m for the 2-layer soil profiles, such as in Figure 4, which were controlled by the strength of the weaker liquefied soil. The authors have successfully calibrated a limit equilibrium method to predict $M_{max}$ in some of these 3-layer pile centrifuge models, after incorporating basic kinematic considerations to allow for the change in pile curvature (and hence of the sign of the passive soil pressure on the pile) within the top nonliquefied soil layer.

The comparisons in Figures 10 and 11 between Models 2 and 2m reveal interesting aspects of the role played by superstructural inertia in the lateral spreading process. For depths greater than 2 or 3 m, the effect of lateral spreading predominates and the inertial loading due to the mass can be ignored. However, at shallow depths of less than 2 m, that is in the top nonliquefiable layer, the bending moments of the two centrifuge models are very different, with those of Model 2m changing rapidly with time due to the combined effect of inertia and lateral spreading. However, even in Model 2m the maximum moments still tend to concentrate at the upper and lower boundaries of the liquefied layer. Despite the rapid change in shallow bending moments due to the mass, when the top soil layer failed in passive in Model 2m, the pile head and cap "snapped" in the downslope direction, exactly the same as in Model 2 (Figure 11), showing that the soil failure mechanism was still controlled by lateral spreading.

Another factor which has been studied in the centrifuge for the 3-layer soil model is the influence of the superstructural stiffness that field case histories has shown to be important. This has been done by the addition of lateral and rotational springs above ground connected to the pile head, such as spring k in Figure 12 (Ramos, 1999). As expected, the analysis of these centrifuge results has required significant kinematic considerations and parameters, even when simple limit equilibrium calculations are conducted. On the other




■ Figure 11. Snapping of pile in downslope direction in centrifuge models without (Model 2) and with (Model 2m) inertial loading (Wang, 2001)

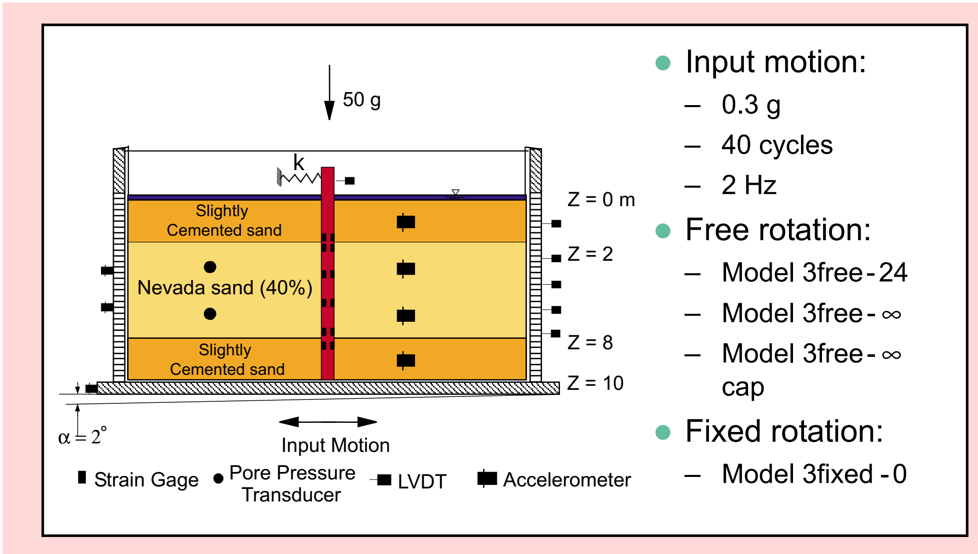



Figure 12. Lateral spreading pile centrifuge model incorporating effect of superstructural stiffness (Ramos, 1999)

hand, some aspects of the analysis become simpler compared with the case of k = 0 (Figure 3), in that if the value of k is large enough, there is no double curvature of the pile at very shallow depths, and no "snapping" of the pile in the downslope direction as in Figure 11. That is, the constraining effect of spring k forces the lateral pressure of the nonliquefied layer on the pile to act in the same downslope direction at all depths between 0 and 2 m.

## Pile Retrofitting Strategies and Results

Both case histories and centrifuge models have shown the great importance of the shallow nonliquefiable soil in increasing the bending response of the pile foundation. Therefore, a promising rehabilitation approach of existing foundations is to replace the shallow soil in a trench around piles and pile cap by a frangible material that will yield under constant lateral soil forces (Figure 13a). This would decrease both bending moments and foundation deformations while allowing the ground lateral spreading to take place without interference from the foundation. As this retrofitting scheme also decreases the lateral resistance of the foundation to inertial loading, the desired frangible material selected, while yielding to static force should remain resilient under the transient inertial loading. Alternatively, the trench surrounding the foundation with frangible material may be located at some distance from the foundation so as to increase the resistance to inertial loading (Figure 13b).

A series of centrifuge models of a single pile with pile cap in the 3-layer soil profile were conducted using the retrofitting setups of Figure 13, labeled respectively Strategy 1 and Strategy 2. These experiments are listed in Table 1, which include also the benchmark nonretrofitted Models 2 and 2m,

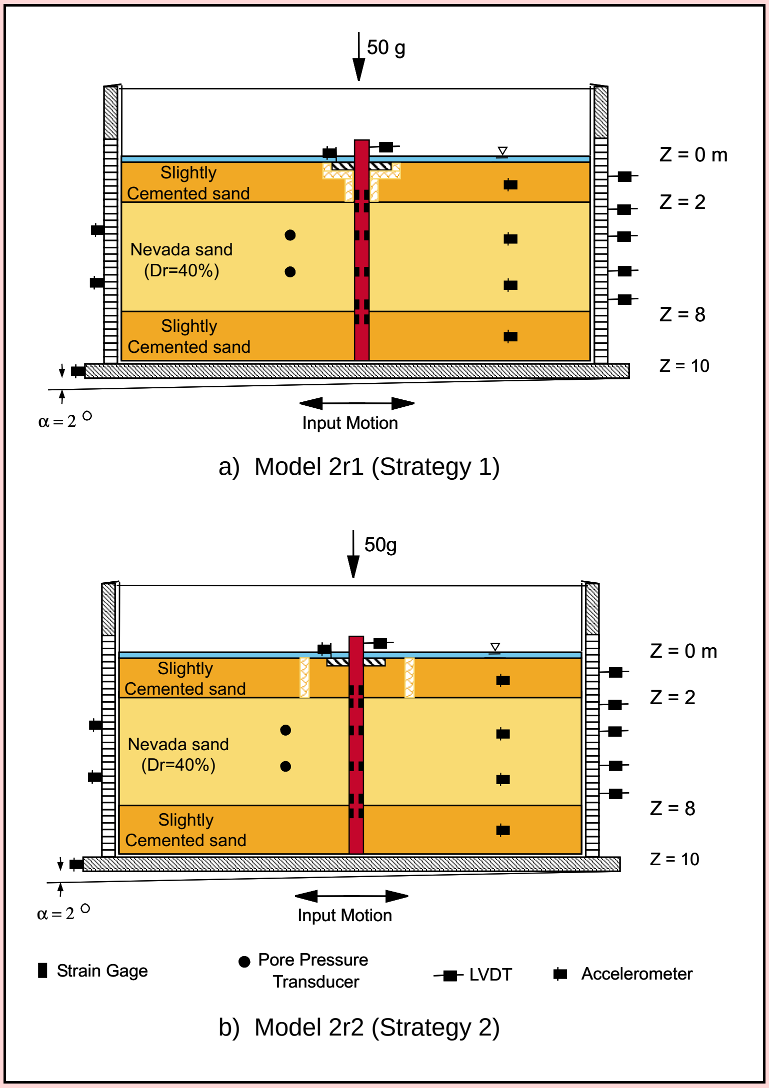
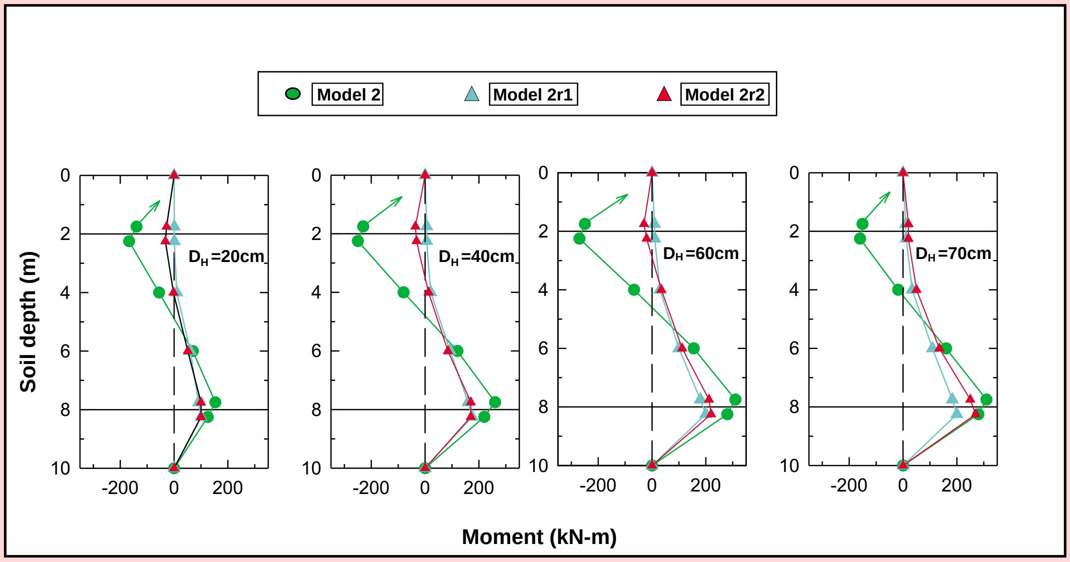



Figure 13. Lateral spreading pile centrifuge models to evaluate retrofitting strategies (Wang, 2001)

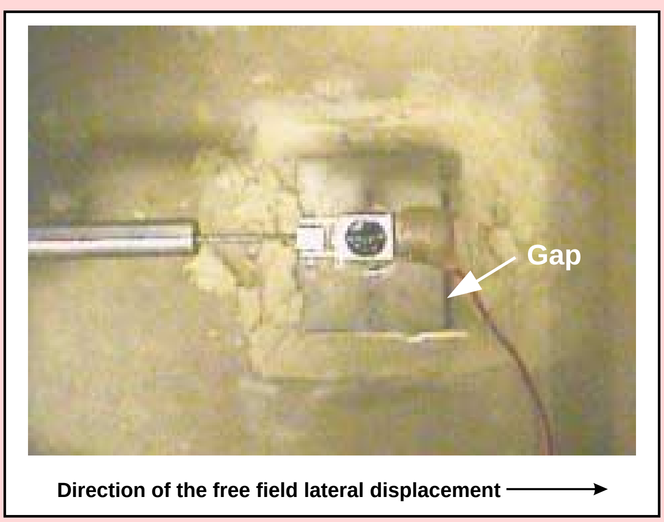
already discussed. Models 2r1, 2mr1a and 2mr1b were done with Strategy 1, without and with a mass above ground, and Models 2r2 and 2mr2 were conducted with Strategy 2. In both cases, a soft clay was placed in a trench either directly around or at some distance from the foundation. In future tests the use of an artificial frangible material with higher resistance to transient loading is planned (Wang, 2001).

Figures 14 and 15 illustrate measurements and observations obtained from Models 2r1 and 2r2. The free field lateral ground displacements during shaking in centrifuge tests without and with pile foundation retrofitting were essentially the same (Figure 11), consistent with the assumption that they represent truly free field response. Figure 14 compares the bending moment response without and with retrofitting. As expected, there




■ Figure 14. Measured bending moment response along pile in lateral spreading centrifuge models without (Model 2) and with (Models 2r1 and 2r2) foundation retrofitting (Wang, 2001)

is a dramatic reduction in the moments in the top 2 m of pile in contact with the nonliquefiable soil. The maximum moment there was close to 300 kN-m in Model 2 and becomes about 10 kN-m after retrofitting.A smaller reduction is also observed for the maximum bending moment at the lower boundary of the liquefied layer, at about 8 m depth. Similarly, the pile head displacements at the end of the tests were reduced by a factor of two by retrofitting (from 85 to 40-50 cm, with $D_H$ = 70 to 80 cm for the soil in the free field). The photo of Model 2r1 in Figure 15, taken after the test, illustrates the corresponding "crunching" of the soft clay against the pile cap in the upslope side, and opening of a gap downslope between soil and foundation. However, the counterpart to this reduction in permanent bending response to lateral spreading of the pile foundation was an increase of transient pile accelerations and displacements, especially in the tests incorporating inertial loading (Models 2mr1a,b and 2mr2, not shown), due to the reduced lateral ground support in the top 2 m of the foundation; future tests will address this problem.




■ Figure 15. Plan view of retrofitted pile cap and ground after the test, Model 2r1 (Wang, 2001)

**Table 1.** Program of Centrifuge Tests to Evaluate Retrofit Strategies 1 and 2 (Wang, 2001)

| Test No. | Cap | Mass | Retrofitting | Comments |
|---|---|---|---|---|
| 2 | Yes | No | No | |
| 2r1 | Yes | No | Yes | |
| 2r2 | Yes | No | Yes | |
| 2m | Yes | Yes | No | |
| 2mr1a | Yes | Yes | Yes | |
| 2mr1b | Yes | Yes | Yes | Repeat of 2mr1a |
| 2mr2 | Yes | Yes | Yes | |

## Conclusions and Future Research

Case histories during earthquakes have shown the significance of lateral spreading in causing damage to deep foundations and supported structures during earthquakes.The complexity of the problem requires use of centrifuge physical modeling to clarify mechanisms, quantify relations and calibrate analysis and design procedures. Centrifuge results so far have clarified the deep foundation response, have shown significant agreement with field experience, and are being used to calibrate limit equilibrium and Beam-on-Winkler-Springs (p-y) analytical methods. Specifically, the importance of the shallow nonliquefiable soil layer riding on top of the liquefied soil in increasing foundation bending response has been clarified. Retrofitting strategies are being evaluated in the centrifuge, aimed at mitigating the effect of lateral spreading associated with the pressure of this shallow layer while preserving needed lateral resistance to inertial loading. Additional work is needed to understand and quantify the response of nonretrofitted and retrofitted pile foundations, with centrifuge model experiments combined with case studies and theory, toward improving the state-of-practice of seismic design and retrofitting of deep foundations against liquefaction.

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