The 14th World Conference on Earthquake Engineering
October 12-17, 2008, Beijing, China

STUDY OF THE BEHAVIOR OF PILE GROUPS IN LIQUEFIED SOILS

Shin-Tower Wang[1], Luis Vasquez[2], and Lymon C. Reese[3], Honorary Member,, ASCE

[1&2] *President & Project Manager, Ensoft, Inc. Email: ensoft@ensoftinc.com*
[3] *Professor Emeritus, Dept. of Civil Eng., The Univ. of Texas, Austin, Texas. Email: lymonreese@aol.com*

ABSTRACT :

One of the most dramatic causes of damage to structures during earthquakes has been the development of liquefaction in deposits of loose and saturated sand. Vibration due to the earthquake causes settlement of the ground surface and the resulting upward flow of water frequently turns the sand into a "quick" or liquefied condition. When liquefaction takes place during earthquakes, the ground often undergoes a large amount of permanent deformation as a result of lateral movement, even though the ground is nearly flat. Technical literature contains little information on quantifying the magnitude of the lateral spreading against piles in a group. This paper investigates the behavior of pile groups in deposits of liquefied soil, which is moving laterally (so called lateral spreading). As a foundation problem, the analysis of a pile foundation under axial and lateral loading is complicated by the fact that the soil reaction is dependent on the pile movement, and the pile movement, on the other hand, is dependent on the nonlinear soil response. If piles are situated in a soil layer undergoing lateral movement, it is evident that horizontal pressures are developed against the piles. The net soil force on piles depends on the relative movement of the pile and soil. This paper introduces an analytical solution to take into account the effect of lateral spreading on the behavior of pile groups during the post-liquefaction period. The configuration of the piles in a group can be an important factor in reducing the force from lateral spreading, which is identified in this paper.

KEYWORDS: lateral spreading, nonlinear soil, p-y curves, liquefied soil, pile groups

1. INTRODUCTION

The factors affecting liquefaction of sands have been extensively investigated in the past four decades. The understanding of the phenomenon has advanced to a degree that analytical procedures have been formulated to predict liquefaction at a particular site. The lateral resistance of deep foundations in liquefied sand is often critical to the design of bridges and buildings. When liquefaction during earthquakes takes place in sandy deposits, the ground is known to undergo often a large amount of permanent deformation as a result of lateral flow of liquefied soils, even though the ground is nearly flat. If the free-field soil movement is greater than the pile displacement, then spreading soils will apply extra driving pressure on the already loaded piles. The actual force on the pile induced by the soil movement is dependent on the relative displacement between the pile and soil. If the liquefied soil causes the upper layer to become unstable and move laterally, a model recommended by Wang and Reese (1998) may be used to solve for the behavior of piles and the pile group.

This paper investigates the behavior of piles in a group in deposits of liquefied soil, which is moving laterally. Prior to a discussion of the ability of piles to sustain lateral loading, a discussion is given on the liquefaction of soils and the way to estimate the residual strength of such liquefied soils.

2. MODELING OF THE PILE-SOIL SYSTEM

As a foundation problem, the analysis of a pile foundation under axial and lateral loading is complicated by the fact that the soil reaction is dependent on the pile movement, and the pile movement, on the other hand, is dependent on the soil response. Thus, the problem is one of soil-structure interaction. The problem is nonlinear because soil response is a nonlinear function of pile displacement. To represent the nonlinear response of soil behavior, a discrete model is commonly-used for piles and the soil interaction as indicated in Fig. 1. The discrete curves of soil response for each depth may not faithfully model the true continuum; however, the curves are firmly based on the results of many full-scale experiments. The discrete model has the merit of allowing engineers to specify various soil properties at each increment along a pile. Recommendations for computing load-transfer curves used in the discrete pile model are in the technical literature, derived from experiment and analysis. Generally, the model for piles under lateral loading employs *p*-y curves for lateral soil resistance. The load-transfer model for piles under axial loading employs *t-z* curves (side friction) and *q-w* curves (tip resistance) to represent the response of the soil. Similar load-transfer model can be used for torsional loading although the torsional components generally are very small in the pile group.

Analytical solutions have been developed for analyzing the large deformation of piles in a group subjected to loading in three-dimensional space. The computer program (GROUP v7) was developed based on the discrete model to compute the distribution of loads (vertical, lateral, and overturning moment) from the pile cap to individual piles in the group. The piles may be installed vertically or on a batter and their heads may be fixed, pinned, or elastically restrained by the pile cap. The cap may settle, translate, and rotate and is assumed to act as a rigid body. The model uses internally the nonlinear response of the soil, in the form of *t-z* and *q-w* curves for axial loading, t-θ curves for torsional loading, and *p-y* curves for lateral loading. A solution requires iteration to accommodate the nonlinear response of each of the piles. The equations of equilibrium are satisfied, and compatibility is achieved between pile movement and soil response, and between the movement of the cap and the pile-head movement. A pushover analysis can be performed, considering the nonlinear response of the soil, where the failure of one or more of the piles is reflected in excessive movement or excessive bending moment. The model allows the engineer to investigate in a timely manner the comparative responses of groups with differing conditions, such as the effect of reduced soil strength due to liquefaction.

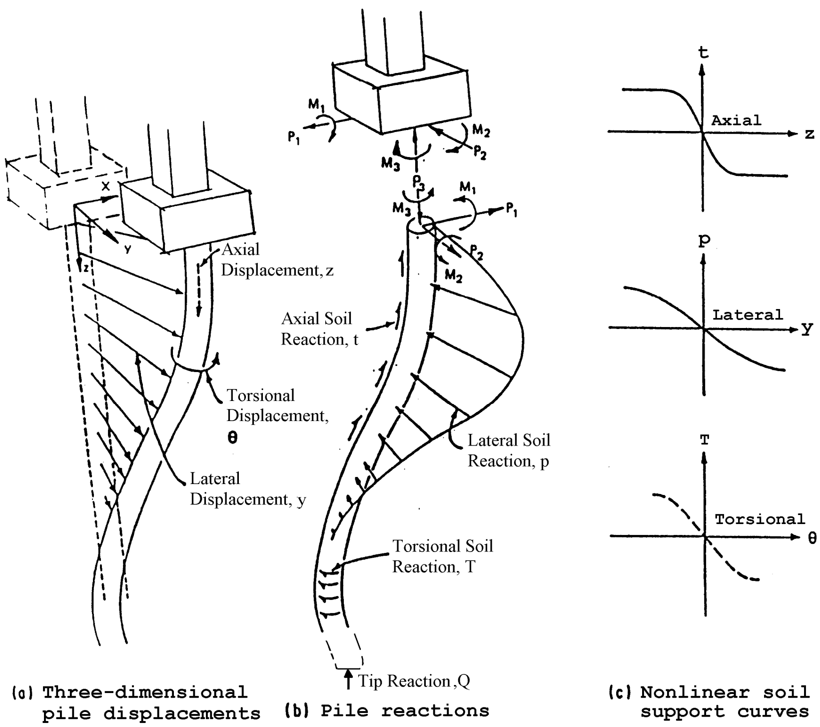

Fig. 1 Discrete model for pile-soil interaction

3. RESPONSE OF LIQUEFIED SAND

When sand is liquefied under seismic condition, some suggest that it behaves similar to soft clay. Seed and Harder (1991) examined documented cases where major sliding had occurred due to liquefaction and where some conclusions can be drawn concerning the strength and deformation of liquefied soil. Seed and Harder recommended that residual strength of about 10% of the effective overburden stress be used for the shear strength of liquefied sand. The behavior of the piles or drilled shafts under lateral loading is dependent on pile properties (diameter and flexural rigidity), penetration length, and the lateral resistance of soil layers. Although reasonable methods have been developed to define p-y and t-z curves for non-liquefied sand and other soil types, considerable uncertainty remain regarding how much soil resistance can be provided by liquefied strata. In many cases, liquefied sand is assumed to have no lateral resistance and the assumption in some cases may be conservative (Rollins et al 2005). Wang and Reese (1998) have studied the behavior of piles in liquefied soil by treating the liquefied sand as soft clay. The p-y curves were generated using soft clay criteria and the cohesive strength (c) is based on the residual strength of liquefied sand. The strain-control parameter (e50) of 0.05 was used in their studies. Although simplified methods have been used for design based principally on engineering judgment, full-scale field tests are needed to develop a full range of p-y curves for liquefied sand.

Rollins et al (2005) have presented recent results on lateral resistance of a full-scale pile group in liquefied sand. Based on their studies, p-y curves in liquefied sand are characterized by a concave-up load displacement shape where the slope of the curve increases as displacement increases as shown in Fig. 2. This characteristic concave-up load displacement shape appears to result primarily from dilative behavior during shearing, although gapping effects may also contribute to the observed response. Rollins and his co-workers also found that p-y curves for liquefied sand stiffen with depth (or initial confining pressure). With increasing depth, smaller displacement is required to develop significant resistance, and the rate at which resistance develops as a function of displacement also increases. Following liquefaction, p-y curves in sand become progressively stiffer with time as excess pore water pressure dissipates. The shape of a p-y curve appears to transition from concave-up to concave-down as pore water pressure decreases. An equation based on results of experiments has been developed by Rollins (2005) to describe mathematically the observed load-displacement response of fully liquefied sand as a function of depth. Based on results of studies, p-y curves developed with the Rollins equation shown below have been used in many recent case studies. The derived equation is a simple mathematical form and is given in Eq. 1.

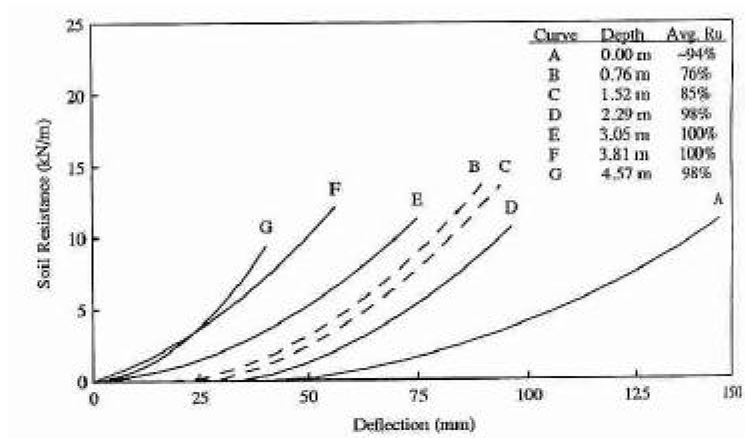

Fig. 2 Soil resistance (p-y) curves recommended by Rollins et al (2005)

$$p = A\,(B\,y)^{C} \qquad (1)$$

in which $A = 3 \times 10^{-7}\,(z+1)^{6.05}$; $B = 2.80\,(z+1)^{0.11}$; $C = 2.85\,(z+1)^{-0.41}$; p is soil resistance (kN/m); y is lateral deflection of pile (mm); and z is depth (m).

Application of Eq. 1 should generally be limited to conditions comparable to those from which it was derived, namely for soil pressure of 15 kN/m or less, deflection of 150 mm or less, depth of 6 m or less, and sands with initial relative densities of approximately 50%. The equation was also derived from subsurface conditions in which the water table was very near the ground surface. Consequently, the parameters of depth and initial vertical effective stress were almost directly proportional. In other design situations, subsurface conditions may be different. In such cases, the depth variable (z) may be equated with the initial vertical effective stress divided by 10 kN/m^3, which is generally representative of the effective or buoyant unit weight of the sand at the site. Weaver (2001) studied the diameter effects for different sizes of piles and recommended a modification factor for correcting Eq. 1.

4. CONSIDERATION OF LATERAL SPREADING

When liquefaction takes place in sandy deposits during earthquakes, the ground often undergoes a large amount of permanent deformation as a result of lateral flow, even though the ground is nearly flat. Permanent deformation has been observed following the main shock and during a period of no shocks or ones of small-intensity (Bartlett and Youd, 1995). The lateral flow is due to the action of gravity-induced shear stress, which is in excess of the residual shear strength of the liquefied soil. Liquefaction-induced lateral spreading can add large lateral forces on piles which may or may not be able to remain stable. The magnitude of the lateral forces on piles induced by viscous-flow loading is difficult to predict. Relevant factors are properties of the liquefied soil, distribution of the soil displacement, and the velocity of the viscous flow. The technical literature contains little information on quantifying the magnitude of the earth force against piles from the sliding soils.

If a pile is situated in a soil layer undergoing lateral movement, the net soil reaction on piles depends on the relative movement of the pile and soil. If the pile at a particular depth has more displacement than the soil at the same depth, then the soil will provide resistance to the pile as a function of the relative displacement. However, if the free-field soil movement is greater than the pile displacement, then the soil will apply extra driving pressure on the already loaded pile.

For a particular depth, the soil resistance versus the pile deflection for a given soil movement is shown in Fig. 3, where y_S defines the free-field movement of the soil. As may be seen, the soil-resistance curve is not symmetric with respect to the y-axis, and the entire curve is shifted according to the amount of soil movement. The whole soil-resistance curve is offset and becomes symmetric to Line AA. The p-y curve in Fig. 3 is drawn in the first and fourth quadrants for convenience; however, p is opposite in sign to y. If the pile deflection y_1 is less than the soil movement y_S at certain layers, the surrounding soil gives the pile a driving force instead of a resisting force, as indicated in Fig. 3. If the pile deflection at some depth is y_2 and the soil movement is again y_S, the soil is now resisting the pile movement. The p-y method has been recognized as a rational method to be used for large-deformation problems. The method can include the free-field soil movement into the soil-pile-interaction analysis with a proper modification in computation. The key point in dealing with the soil movement in the analysis is to find the relative movement between the pile and soil at point to point along a pile.

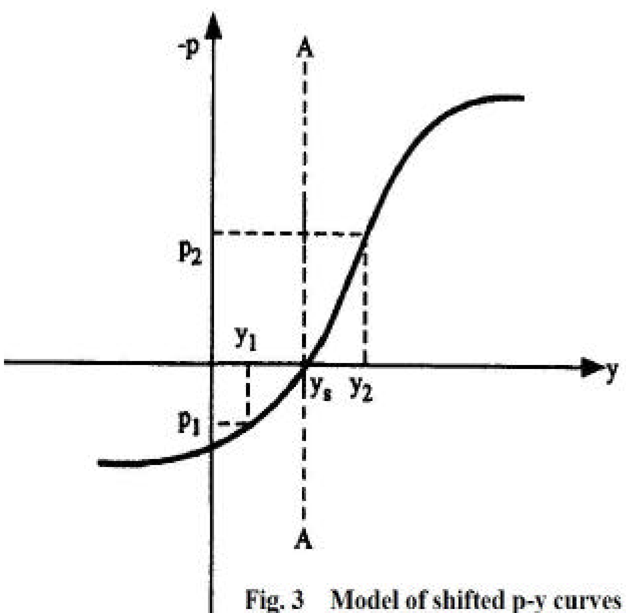

Fig. 3 Model of shifted p-y curves for soil movement

Empirical predictions of liquefaction-induced lateral spreading are available for estimating the movement of a soil mass as a result of a seismic event (Bartlett and Youd, 1995). If such an estimate reveals that the movement of soils is greater than 150 mm, the driving force on the pile from the soil, assuming no pile movement, then will almost reach the maximum value or approximately equal to the ultimate p of the p-y curve. As explained earlier, the actual force on the pile is dependent on the relative displacement between the pile and soil. The procedures based on relative displacement, can be implemented to predict the net earth pressure on piles and to investigate the stability of the pile foundation. Further, the assumption is made that the speed of soil movement is relatively slow.

5. CASE STUDIES

Lateral-load tests were conducted at Treasure Island, California, to study the behavior of a pile group following blast-induced liquefaction (Rollins et al, 2005). The test piles were 0.324 m O.D. steel pipes with a 9.5 mm wall thickness and were driven open ended to a depth of approximately 11.5 m below the excavated ground surface. A plan view of the test site during the pile group testing is presented in Fig. 4. The piles in the group were driven in a 3 x 3 pattern with a nominal center-to-center spacing of 3.3 pile diameters in both directions. The load was applied to the load frame at a height of 0.86 m above the ground surface using a 2.2-MN capacity hydraulic actuator powered by an electric pump.

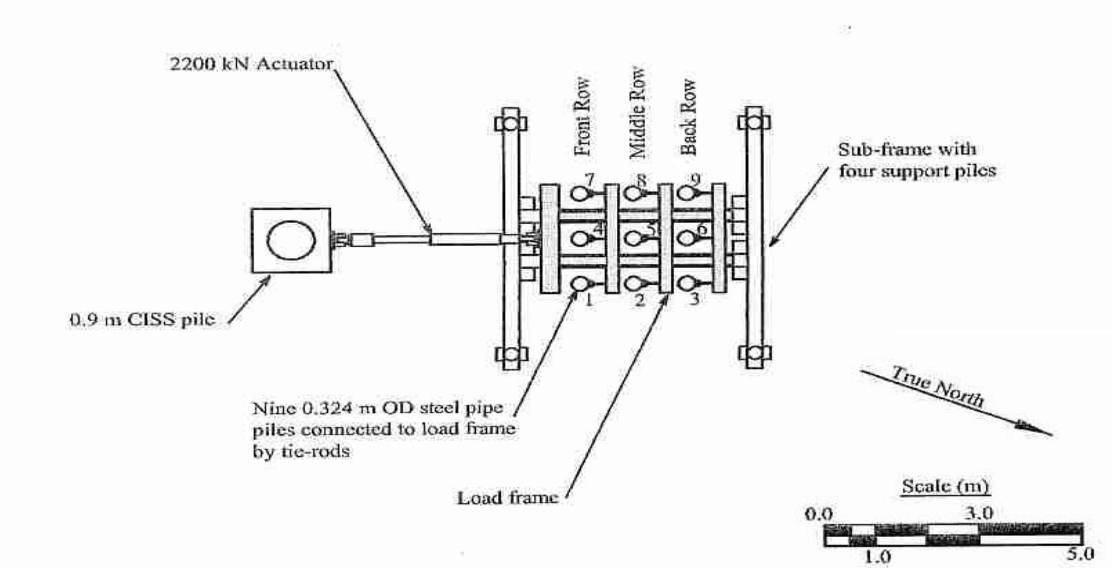

Fig. 4 Plan view of test site during loading tests (after Rollins et al 2005)

The load frame was essentially rigid in comparison with the lateral stiffness of the piles. Load was transferred from the load frame to each pile using a pinned connection. The soil profile consists of hydraulically placed fill and native shoal sands to a depth of about 6 m below the excavated ground surface. The hydraulic fill generally consists of loose fine sand or silty sand underlain by silty sand and Young Bay Mud, as shown in Fig. 5.

A comparison of the bending-moment profiles for the center pile in the front row of the pile group before and after blasting is presented in Fig. 6. The predicted curves for bending moment, based on the soil profile and p-y criteria of liquefied sand recommended by Rollins et al (2005), are presented in the same figure. The agreement between the curves for measured and predicted bending moment, for both before and after blasting, is very good. The behavior of the piles, if lateral spreading near the top of the liquefied soil was developed, was not able to be studied in the field tests. Based on the successful establishment and the excellent prediction of the analytical solution in Program GROUP v7, the model was further expanded to study the moment distribution of the piles in the test group assuming lateral spreading was developed within the top 3 meters. The further assumptions were made that lateral spreading moves excessively and the driving pressure on the piles has reached the maximum value (approximately 15 kN/m). The moment-distribution curves during lateral spreading are also presented in Fig. 6. The critical conditions are that the maximum bending moment not only increased significantly, but also shifted down to the un-liquefied soil layers, which is consistent with the findings in the centrifuge tests by Abdoun et al (2003). Figure 7 presented the moment distribution curves for each pile in a group during lateral spreading. The figure also indicates that the behavior of piles depends on the location of each pile in a group. The shadow-effect is significant for piles in the trailing rows. The arrangement of the pile layout in a group will affect the soil pressure and resistance on each of piles, which can be further investigated by using this numerical tool.

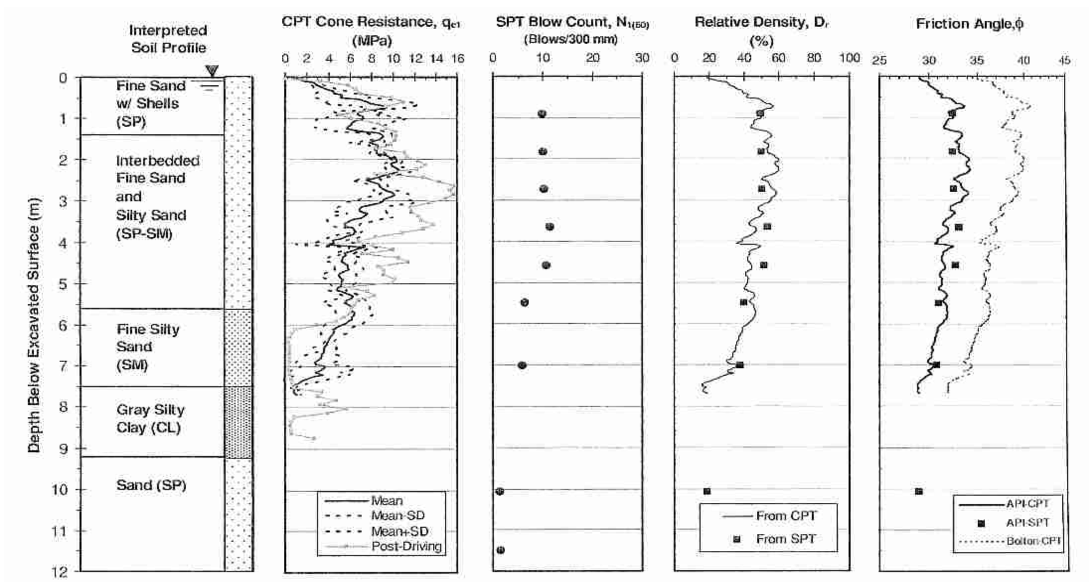

Fig. 5 Soil profile at the test site (after Rollins et al 2005)

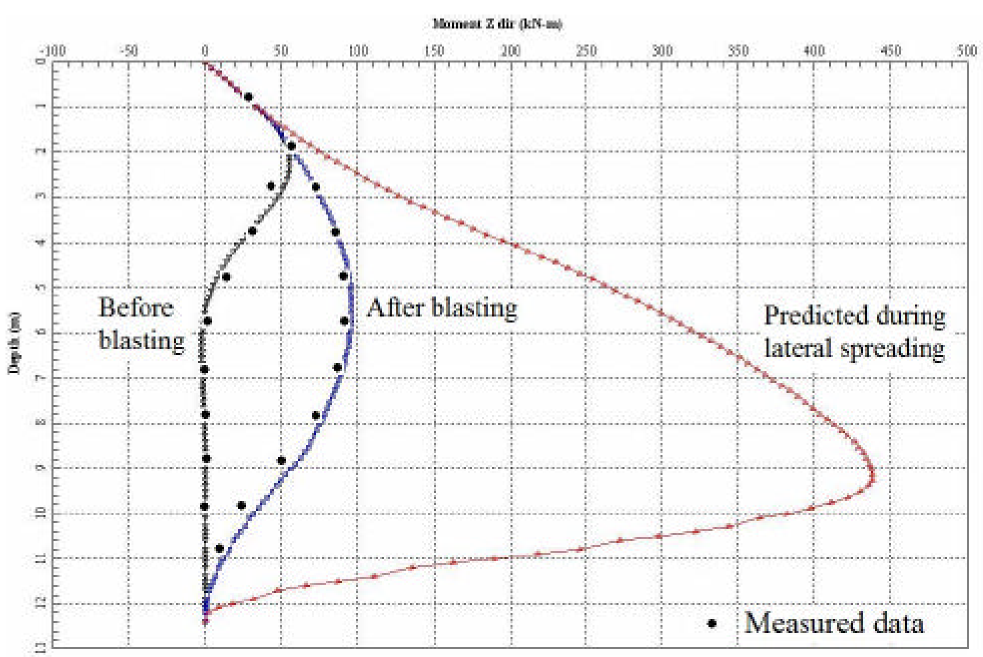

Fig. 6 Bending-moment curves under 30 kN/pile before/after blasting and during lateral spreading

6. CONCLUSIONS

Analytical solutions have been developed for analyzing the behavior of piles in a group subjected to loading in three-dimensional space in nonlinear soil. Recently, field investigation on the behavior of piles in liquefied soil led to the development of p-y curves for soil resistance without lateral spreading. The development of lateral spreading in liquefied soil can cause pile failure in shear and in bending due to the unexpected large pressures generated by the soil movement. The analytical technique presented in this paper for studying the interaction between soil and piles during lateral spreading has proven to be rational and useful. The structural components and the soil models of the analytical method have been coded and iteration is used to accommodate the nonlinear response of the soil around each pile in a group. Parameters can be varied through a wide range and computed results will provide important guidance to the engineer in achieving a reasonable judgment about the effect of lateral spreading on the behavior of the pile group. The weakness in the method lies not in the concept or in the ability to make the computations but in lack of sufficient field data for further improvement on p-y curves for liquefied soil in deeper layers.

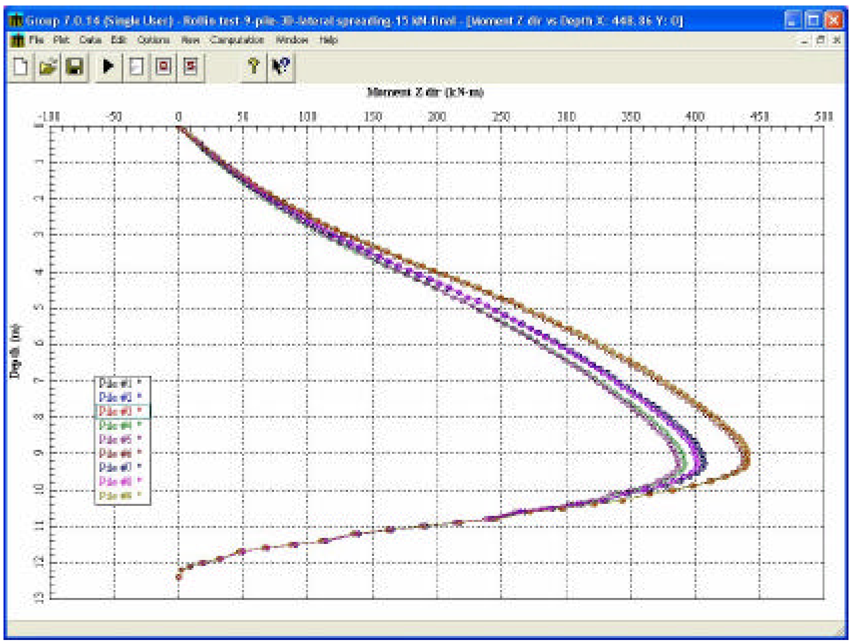

Fig. 7 Bending moment curves for each pile during lateral spreading (30 kN/pile)

REFERENCES

Abdoun, T., Dobry, R., O'Rourke, T., and Goh, S.H. (2003), "Pile response to lateral spreads: centrifuge modeling," *J. of Geotech. and Geoenviron. Engineering*, ASCE, 129(10).

Bartlett, S. F. and Youd, T. L. (1995), "Empirical Prediction of Liquefaction-Induced Lateral Spread," *J. of Geotechnical and Geoenvironmental Engineering*, ASCE, 121(4)

GROUP 7.0 for Windows, (2006). Analysis of a Group of Piles Subjected to Axial and Lateral Loading, Ensoft, Inc., www.ensoftinc.com.

Rollins, K.M., Gerber, T.M., Lane, J.D., and Ashford, S.A. (2005), "Lateral Resistance of a Full-Scale Pile Group in Liquefied Sand," *J. of Geotech. and Geoenviron. Engineering*, ASCE, 131(1).

Seed, R.B. and Harder, L.F. (1991). "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength," *H. Bolton Seed, Memorial Symposium proceedings*, Vol. 2, edited by J. Duncan, BiTech Publishers, ltd, Vancouver, Canada..

Wang, S.T. and Reese, L.C. (1998). "Design of Pile Foundations in Liquefied Soils," *Geotechnical earthquake Engineering and Soil Dynamic III, GSP No. 75*, P. Dakoulas and M. Yegian, eds., Vol. 2, ASCE, Reston, Va.

Weaver, T.J. (2001). "Behavior of Liquefying sand and CISS Piles During Full-Scale Lateral Load Tests," Ph.D. dissertation, Univ. of California, San Diego.