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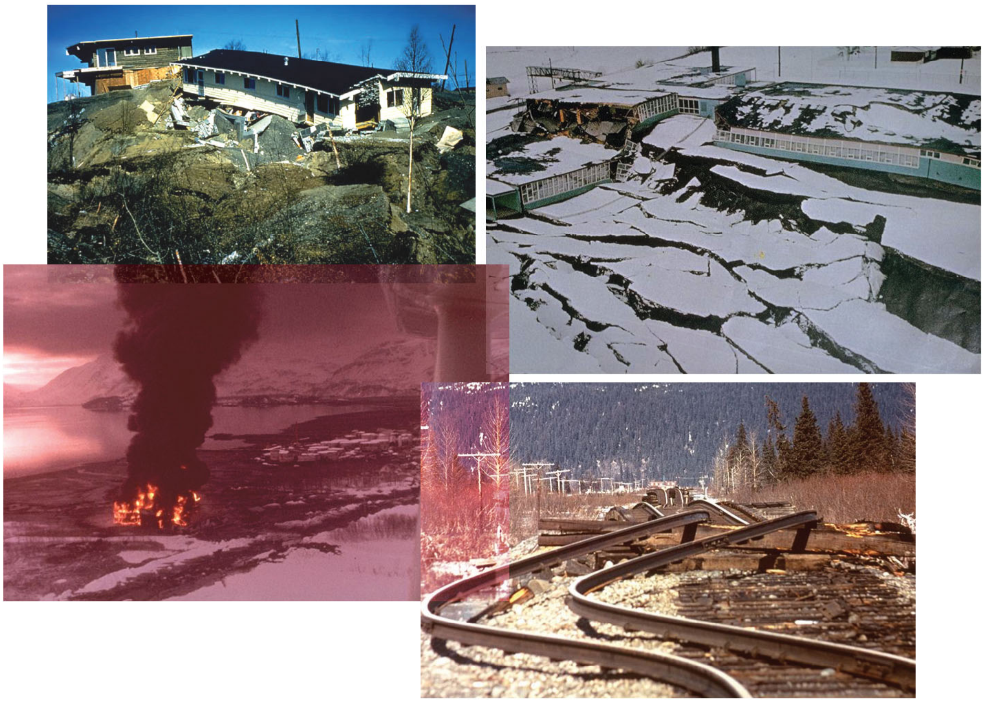

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Extended Abstract for the Presentation:

p-y Behavior in Liquefied & Laterally Spreading Ground in Centrifuge Tests

by Scott J. Brandenberg[1], Ross W. Boulanger[2], & Bruce L. Kutter[2]

This workshop presentation describes aspects of p-y behavior in liquefied and laterally spreading ground in centrifuge tests. The p-y behavior between piles and liquefied sand was back-calculated from centrifuge tests involving: (1) single piles in a level profile of loose sand over dense sand, (2) single piles in a sloping profile of clay over loose sand over dense sand, and (3) a pile group in a sloping profile of clay over loose sand over dense sand. Typical results are presented to illustrate the various factors that influence the magnitude and characteristics of the p-y behavior in the liquefying sands during earthquake shaking.

The tests described herein were performed on the 9-m radius geotechnical centrifuge (Slide 2). Results are presented in prototype units unless otherwise noted. Experimental results are archived for public distribution at the web site for the Center for Geotechnical Modeling (***http://cgm.engr.ucdavis.edu***).

Single Piles in Level Profile of Liquefying Sand

This first series of tests by Wilson et al. (2000) involved simple structures supported on both single piles and pile groups, embedded in a profile of loose sand over dense sand. A schematic cross-section showing only the single-pile-supported structure is shown in Slide 3. The tests were performed at a centrifugal acceleration of 30 g and used a methyl cellulose-water mixture as the pore fluid. Details of the tests and the p-y back-calculation procedures are described in Wilson et al. (2000).

Examples of the back-calculated p-y behavior in the loose sand as it liquefies during shaking are shown in Slide 4. The p-y behavior has characteristics that are consistent with the stress-strain response of liquefying sand, as illustrated by the typical p-y loops. The p-y resistance of loose sand ($D_r \approx 40\%$) was much smaller and softer than for medium-dense sand ($D_r \approx 55\%$). The ultimate lateral resistance in loose sand ($D_r \approx 40\%$) was generally small when the soil liquefied, even when relative displacements (y) were fairly large. In medium-dense sand ($D_r \approx 55\%$), the p-y behavior progressively softened with time during shaking as pore pressures, strains, and number of load cycles increased. The observed p-y behavior was found to be displacement hardening when relative displacements approached or exceeded past values, especially near the surface. This behavior may be attributed to the nearly undrained loading conditions and the tendency for the soil to dilate under these loading conditions (i.e., large enough strains to move the sand through a phase transformation). Similar observations of p-y behavior have since been

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reported by Ashford and Rollins (2002, in press) based on the blast-induced liquefaction testing at Treasure Island and by Tokimatsu et al. (2001) based on large shaking table tests.

Single Piles in Laterally Spreading Profile with Clay Crust over Liquefying Sand

Another centrifuge test involved single pipe piles with diameters of 0.36 m, 0.73 m, and 1.45 m, and one group of two 0.73-m-diameter pipe piles (with an above ground cap connection for fixed head conditions), located at four separate locations in a model slope (Singh 2002). The cross-section in Slide 5 is for the west side of the container and so only shows two of the piles. The test was performed at a centrifugal acceleration of 38g. The soil profile gently sloped toward a channel at one end, and consisted of a nonliquefiable crust of clay (C_u = 23 kPa) overlying a layer of loose saturated sand ($D_r \approx 35\%$), overlying dense sand ($D_r \approx 85\%$).

An example of the recorded behavior during a simulated earthquake event is illustrated by the various time histories in Slide 6 for the soil and the 0.73-m diameter pile. The base motion (plotted at the bottom of the Slide) was a scaled version of a recording from Port Island during the 1995 Hyogoken-Nambu earthquake. The red circles identify the time at which the peak bending moment occurred in the pile, while the blue triangles identify the time at which the peak pile head displacement occurred.

First, some observations on the p time histories in the clay and loose sand.
- The peak p values within the clay layer agreed well with the values predicted using the monotonic p_{ult} values by Matlock (1970); Note that the undrained shear strength was adjusted for the effects of loading rate (considering both the earthquake and centrifuge scaling).
- The peak p values within the loose sand significantly exceeded the JRA approximation of $p = 0.3\sigma_v D$, where σ_v is the total overburden stress and D is the pile diameter. The peak p values during strong shaking all coincided with transient drops in the r_u in the loose sand (i.e., when the loose sand was going through phase transformation). The largest peak p value occurred early in shaking when the peak excess pore pressure ratios (r_u) had not yet exceeded about 50% in the middle of the loose sand layer.
- The peak p values in the loose sand were closer to the JRA approximation after about $t \approx 13$ s, which corresponded to both the end of the strongest shaking and when the r_u values had reached stable high levels.

The peak bending moment for the pile occurred just beneath the interface of the loose and dense sand layers at about $t \approx 8$ s. This peak bending moment coincided with:
- The r_u in the loose sand transiently dipped to a local minimum less than 0%, despite having been up to about 50% immediately beforehand;
- The peak p occurred in the clay, and acted down-slope (positive p);
- The peak |p| occurred in the loose sand, and acted up-slope (negative p); and
- The transient movement of the clay crust was at a local down-slope maximum.

The peak displacement of pile head occurred at about $t \approx 12$ s, and coincided with:
- The r_u in the loose sand transiently dipped to a local minimum of about 30% despite having been up to about 90% immediately beforehand;
- The p in the clay was at a local maximum that was a little smaller than its past peak value, and again acted down-slope (positive p);
- The |p| in the loose sand was at a local maximum that was a little smaller than its past peak value, and it again acted up-slope (negative p); and
- The transient movement of the clay crust was at a local down-slope maximum.

The interrelations between some of these responses are illustrated in Slide 7.

- The p values in the clay crust and the loose liquefied sand are almost linearly related with opposing signs; down-slope loads from the clay crust are almost always associated with upslope resistances from the loose sand layer.
- The plot of r_u versus p at the middle of the loose sand resembles a q-p′ plot for cyclic stress-controlled loading of a saturated sand. The highest r_u values in the free-field are associated with small |p| values in the loose sand, whereas transient dips in r_u (phase transformation) are associated with the peak |p| values in the loose sand.

Pile Groups in Laterally Spreading Profile with Clay Crust over Liquefying Sand

This series of tests involved a group of six piles connected together by a pile cap and embedded in a model slope. The piles were 0.73-m diameter and spaced at four diameters center-to-center. The cross-section in Slide 8 illustrates one of these tests (Brandenberg et al. 2001). Each test was performed at a centrifugal acceleration of 38g. The soil profile gently sloped toward a channel at one end, and consisted of a nonliquefiable crust of clay overlying a layer of loose saturated sand (D_r≈35%) overlying dense sand (D_r≈85%). Variations between the different centrifuge experiments have included different shear strengths for the clay crust, different earthquake characteristics, and different thickness for the loose sand layer.

A photograph of the model surface after the end of testing with the container removed from the centrifuge arm is shown in Slide 9. Ground movements were much larger on the down-slope side of the pile group than on the up-slope side. A gap of about 2-m (prototype) formed on the down-slope side of the pile cap. Large cracks propagated from the uphill corners of the pile cap toward the sides of the container. The ground formed a bulge or mound in front of the pile cap, although a distinct failure plane was not discernable during excavation.

Models were dissected after testing and the location of “markers” were mapped to define the variation of ground deformations with depth. The photograph in Slide 10 shows a side view of a vertical excavation in one of the centrifuge models. The dark soil at the top is the reconstituted Bay Mud layer. A thin horizontal layer of black (stained) sand marks the contact between the underlying loose and dense sand layers. The embedded marker columns showed that strain in the dense sand and clay layers was small, and that strain in the loose sand increased from the loose sand / dense sand interface to the top of the loose sand. A concentrated zone (localization) of deformations occurred at the contact between the clay crust and the underlying liquefied loose sand. This localization, with as much as 1 m of offset across it, is attributed to void redistribution that occurs as the clay layer impedes the upward seepage of pore water, which is driven by the hydraulic gradients produced by the shaking-induced excess pore water pressures in the underlying sands.

An example of the recorded behavior during a simulated earthquake event is illustrated by the various time histories in Slide 11. Again, the base motion (plotted at the bottom of the Slide) was a scaled version of a recording from Port Island during the 1995 Hyogoken-Nambu earthquake. The red circles identify the time at which the peak bending moment occurred in the pile, while the blue triangles identify the time at which the peak pile head displacement occurred.

First, some observations on the time histories for p in the loose sand and for the lateral loads from the clay crust above the shear gages near the pile heads.

- The passive load from the clay crust exceeded the value predicted using Rankine passive earth pressure theory, but was slightly smaller than the value predicted using a Mononobe-Okabe approach. Friction between the sides of the pile cap and the clay, friction between the base of the pile cap and the clay, and loading on the pile segments above the shear gages accounted for nearly half of the lateral load from the clay crust.
- The peak p values within the loose sand significantly exceeded the JRA approximation of $p=0.3\sigma_v D$. The peak p value occurred during a pulse of strong shaking when the r_u transiently dipped below 0% (i.e., when the loose sand was going through phase transformation). Other peak p values also coincided with transient dips in the free-field r_u.
- The peak p values in the loose sand were closer to the JRA approximation after about $t \approx 13$ s, which corresponded to the end of the strongest shaking. The r_u values were still at stable high levels, having reached those levels earlier in shaking.

The largest peak bending moments for the piles occurred at their connections to the pile cap, with a smaller (opposite sign) peak moment occurring just beneath the interface of the loose and dense sand layers. The largest peak bending moment occurred at about $t \approx 8$ s and coincided with:

- The r_u in the loose sand transiently dipped to a local minimum less than 0%, despite having been up to about 90% immediately beforehand;
- The peak lateral load from the clay crust occurred, and acted down-slope (positive lateral load);
- The peak $|p|$ occurred in the loose sand, and it acted up-slope (negative p); and
- The lateral displacements of the clay crust and pile cap were at local down-slope maximums.

The peak pile cap displacement occurred at about $t \approx 12$ s, and coincided with:

- The r_u in the loose sand transiently dipped to a local minimum of about 50% despite having been up to about 90% immediately beforehand;
- The lateral load from the clay crust was at a local maximum that was a little smaller than its past peak value, and again acted down-slope (positive lateral load);
- The $|p|$ in the loose sand was at a local maximum that was a little smaller than its past peak value, and it again acted up-slope (negative p); and
- The transient movement of the clay crust was at a local down-slope maximum.

The interrelations between some of the recorded responses are illustrated in Slide 7.

- The lateral loads from the clay crust and the p in the loose liquefied sand are almost linearly related with opposing signs; Down-slope loads from the clay crust are almost always associated with up-slope resistances from the loose sand layer.
- The peak inertial load of the pile cap was less than about 25% of the peak lateral load from the clay crust.
- The plot of r_u versus p at the middle of the loose sand is more complicated than previously observed for the single pile example. The largest peak p value did coincide with a transient dip in the free field r_u, but there were a couple of local peaks in p that were associated with high r_u levels and a couple cycles with small p values during strong local dips in r_u. The reasons for these patterns are not entirely clear, but one factor worth noting is that the free-field r_u (away from the piles) will differ from the near-field r_u values (in and around the pile group) due to the local dilation caused by relative movement between the soils and the piles.

Summary of Observations and Concluding Comments

The combined findings from prior physical modeling studies, including those referred to above, show that the p-y behavior of liquefied sand depends on the same factors that are known to affect the monotonic and cyclic loading behavior of saturated sands, as should be expected. Differences in p-y behavior observed by different investigators can be largely explained by consideration of the following factors.

- Relative density (D_r).
- Prior displacement (strain) history.
- Excess pore pressure ratio (r_u) in the far field and near field:
  - Magnitude of cyclic stresses & number of loading cycles imposed by ground shaking.
  - Number of loading cycles between the pile & soil.
- Pile installation method.
- Partial drainage and hence loading rate.
- Soil characteristics.
- Pile foundation stiffness.

The complex cyclic p-y behavior of liquefying sand is only crudely approximated in simplified pushover analyses (limit equilibrium or beam on nonlinear Winkler foundation methods) that attempt to envelop the cyclic loading response. The importance of the p-y approximation for liquefied soil depends on the particular loading mechanism. For example, there are cases where a nonliquefied crust can strongly dominate the lateral loads imposed on a pile foundation, rendering the calculated response relatively insensitive to the assumed properties for the liquefied layer. In other cases, the p-y approximation for liquefied soil can be important enough to warrant a greater level of care.

The approach we are evaluating is to consider the first-order effects of r_u, D_r and cyclic loading condition when estimating the p-y behavior of liquefying soil (Singh 2002). The ultimate lateral resistance (p_{ult}) was assumed to vary linearly with the free-field excess pore pressure ratio (r_u). If r_u=0%, then p_{ult} was taken as the drained capacity, although it is recognized that excess pore pressures could develop locally around the pile. If r_u=100%, then p_{ult} was approximated as 9DS, where D is pile diameter and S is the mobilized shear resistance of the liquefied sand as the pile cyclically moves through it. S was estimated using a normalized ratio of S/σ_{vc}', where σ_{vc}' is the vertical consolidation stress. This normalization was adopted because saturated sands exhibit relatively normalized behavior during cyclic and monotonic (up to some level of strain) loading. Note that the JRA approximation corresponds to an S/σ_{vc}' ratio of 0.03 to 0.07. The appropriate S/σ_{vc}' ratio has been found to depend on relative density and several aspects of the loading condition (as discussed previously). In particular, the S/σ_{vc}' ratio logically is larger for denser sands and larger for a strong virgin loading pulse than for numerous smaller cycles of loading. The appropriate S/σ_{vc}' ratio also depends on how it is used in design analyses, and thus additional parametric studies using pushover design methods are continuing for the purpose of evaluating appropriate design guidelines.

Acknowledgments

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References:

Brandenberg, S. J., Singh, P., Boulanger, R. W., and Kutter, B. L. (2001). “Behavior of piles in laterally spreading ground during earthquakes – centrifuge data report for SJB01.” Report No. UCD/CGMDR-01/02, Center for Geotechnical Modeling, Department of Civil Engineering, University of California, Davis.

Japan Road Association (1996). "Specification for Highway Bridges." (in Japanese).

Matlock, H. (1970). “Correlations for design of laterally loaded piles in soft clay.” Proceedings, 2nd Annual Offshore Technology Conf., Houston, Texas, OTC 1204.

Singh, P. (2002). “Behavior of piles in earthquake-induced lateral spreading.” MS thesis, University of California, Davis.

Singh, P., Subramanian, P. K., Boulanger, R. W., and Kutter, B. L. (2000). “Behavior of piles in laterally spreading ground during earthquakes – centrfiuge data report for PDS01.” Report No. UCD/CGMDR-00/05, Center for Geotechnical Modeling, Department of Civil Engineering, University of California, Davis.

Tokimatsu, K., Suzuki, H., and Suzuki, Y. (2001). “Back-calculated p-y relation of liquefied soils from large shaking table tests.” *Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, S. Prakash, ed, University of Missouri – Rolla.

Wilson, D. W., Boulanger, R. W., and Kutter, B. L. (2000). "Seismic lateral resistance of liquefying sand." J. of Geotechnical & Geoenvironmental Engrg., ASCE, 126(10), 898-906.

Slide 1

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p-y behavior in liquefied & laterally spreading ground in centrifuge tests

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June, 2002

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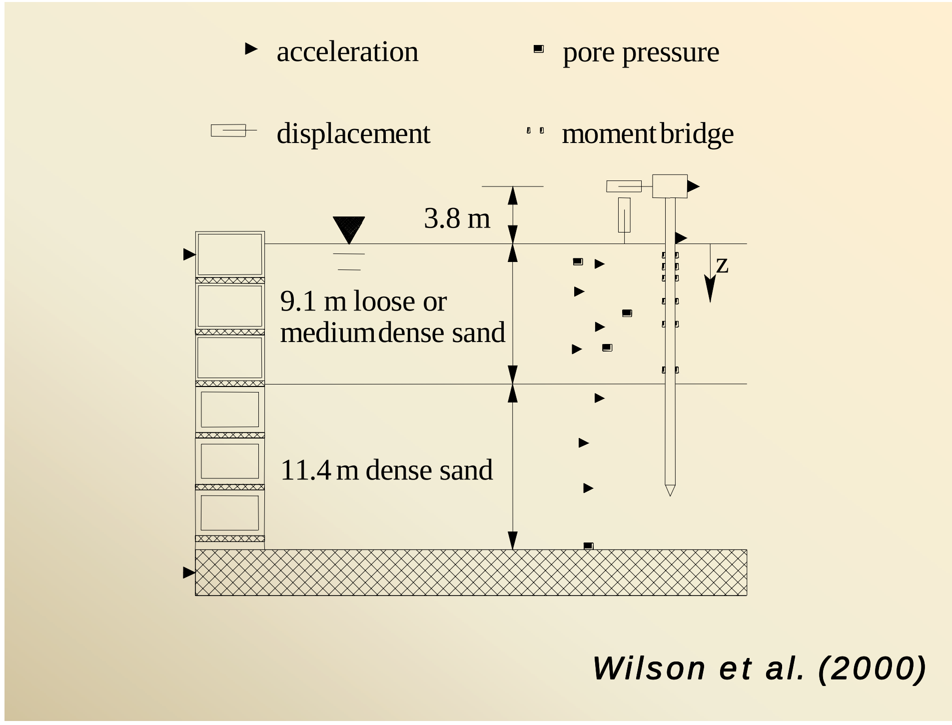
Funding: Caltrans – contract managers A. Abghari, A. Perez

Slide 2

Center for Geotechnical Modeling – University of California, Davis

Slide 3



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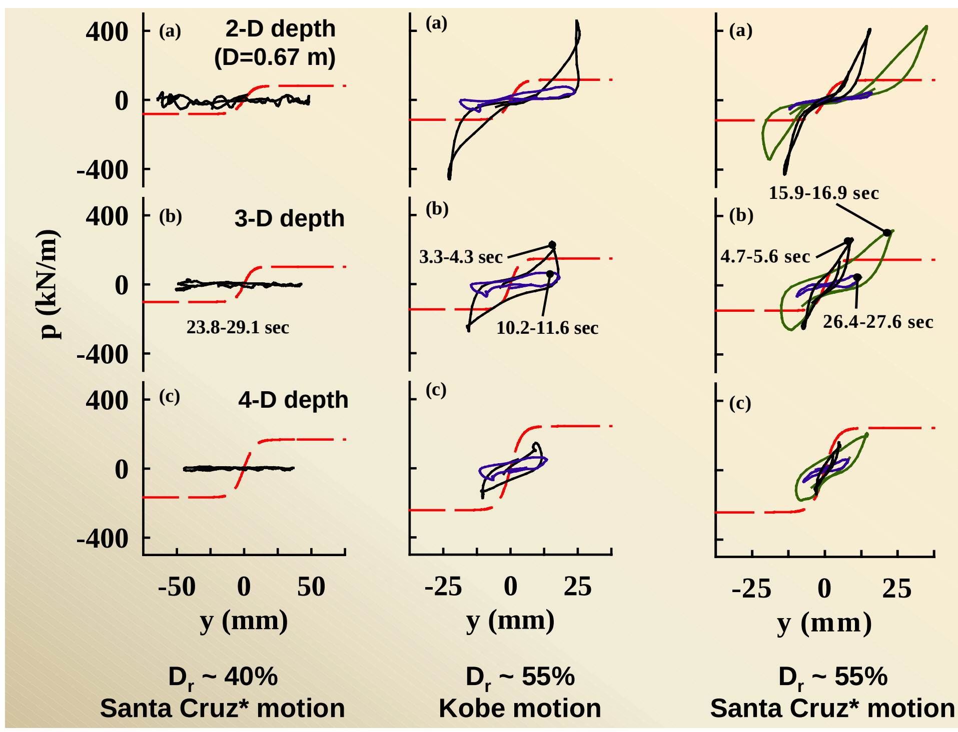

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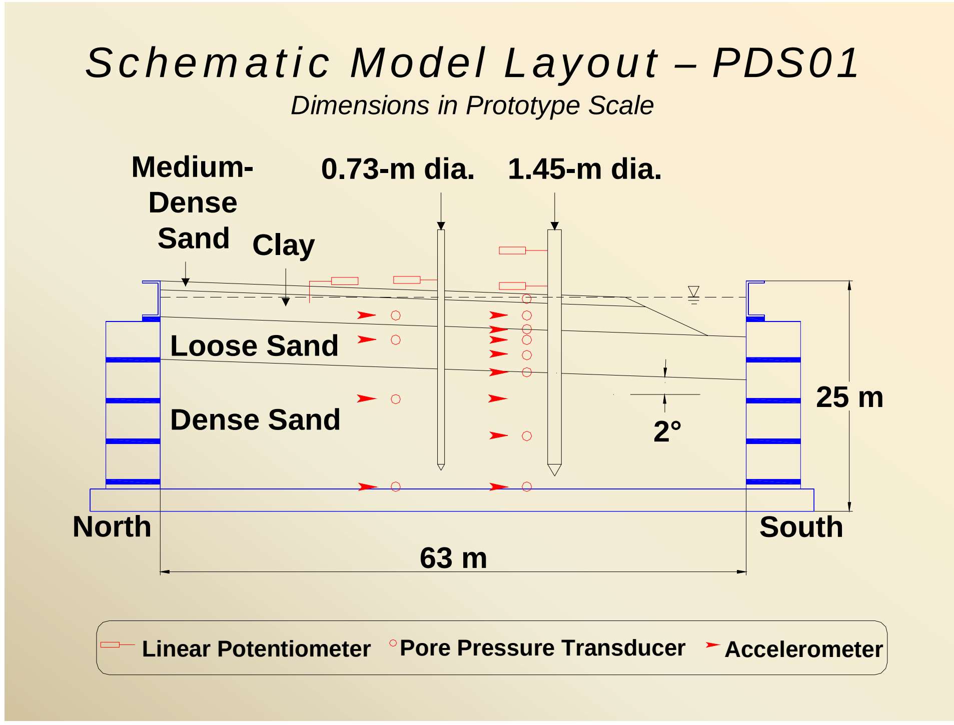

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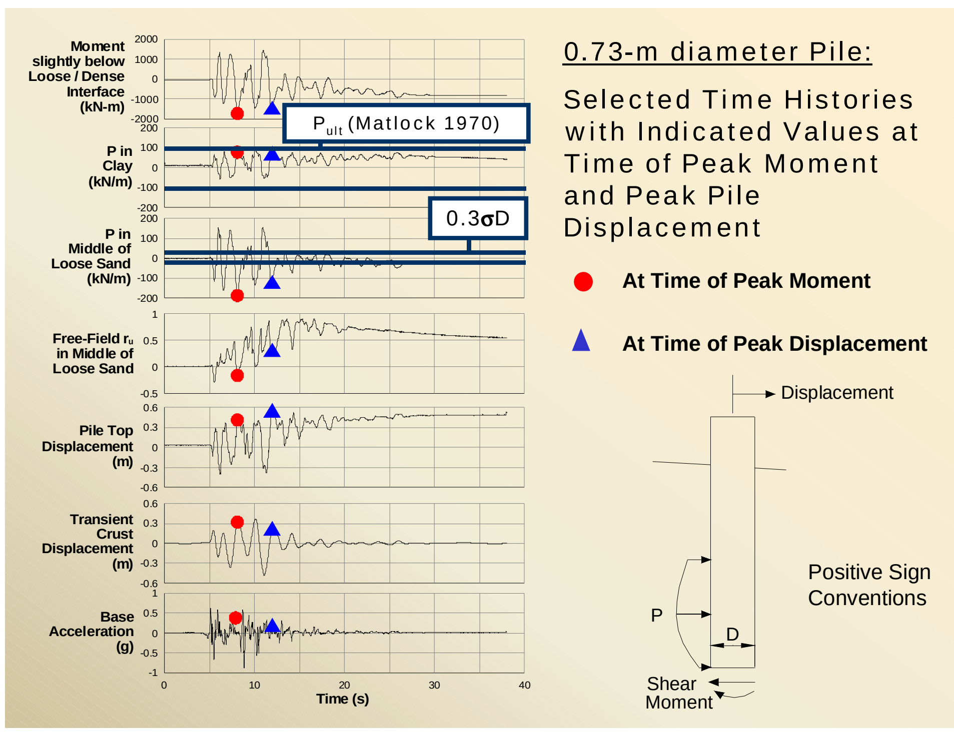

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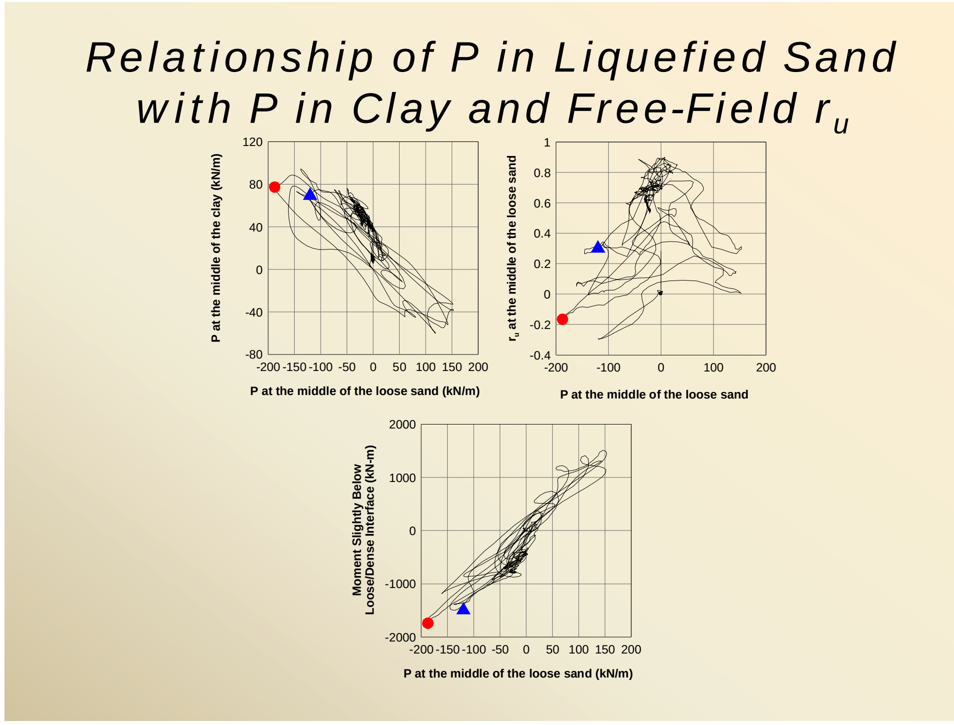

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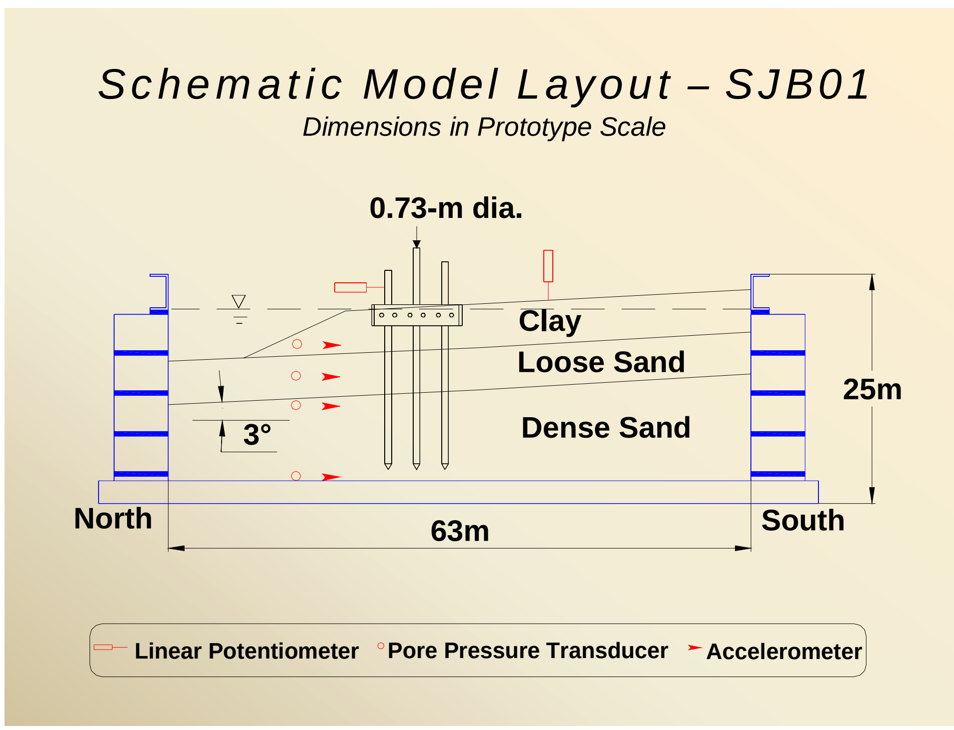
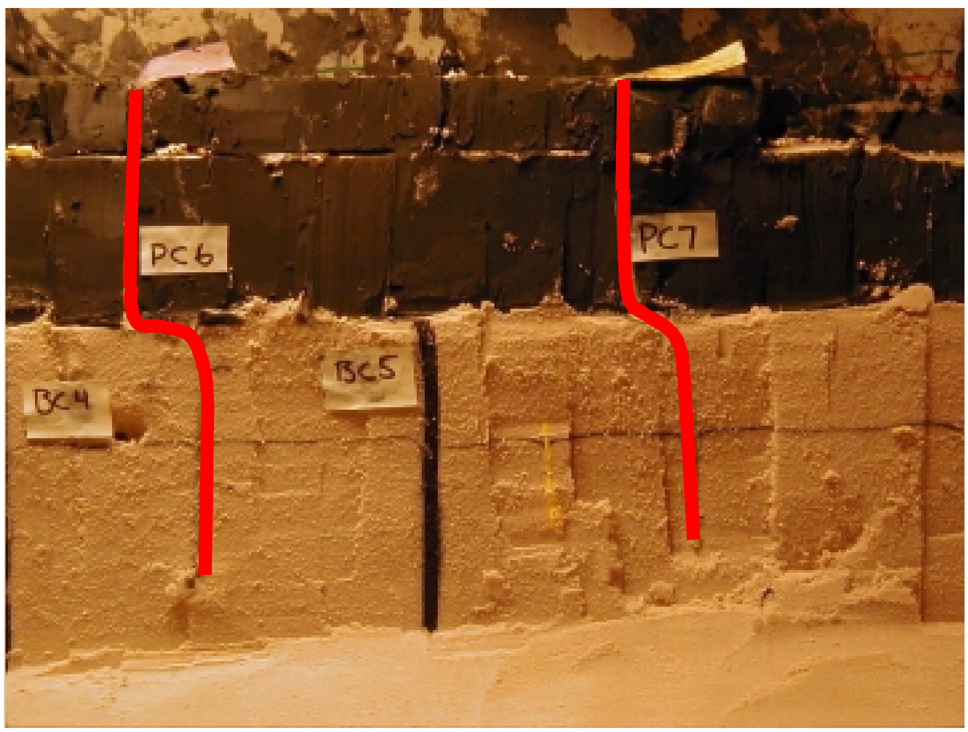


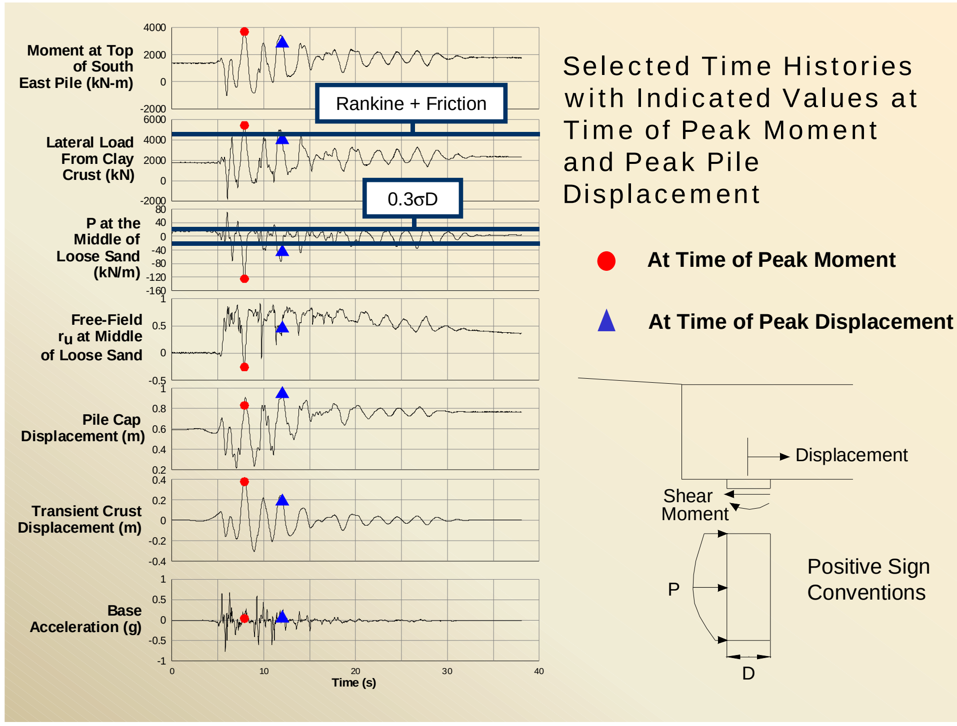
Slide 9

Off the centrifuge after testing:

Slide 10



Slide 11



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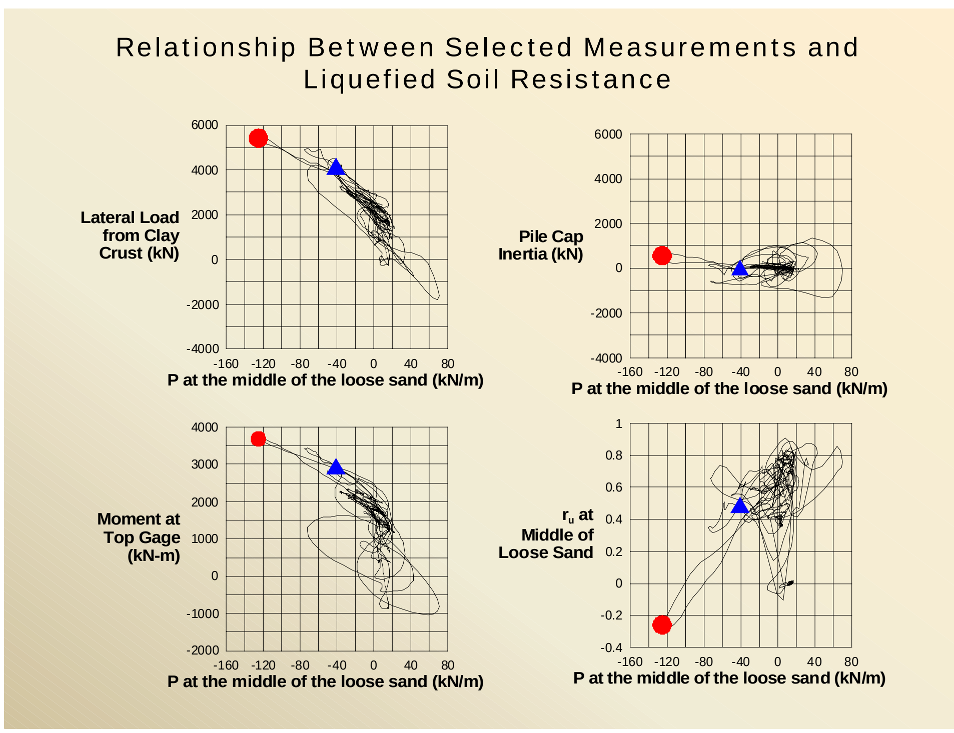

Slide 13

Summary

- ***Physical modeling studies show p-y behavior of liquefied sand depends on the same factors that are known to affect the monotonic and cyclic loading behavior of saturated sands:***
  - ***Relative density (D_r)***
  - ***Prior displacement (strain) history***
  - ***Excess pore pressure ratio in "far field" & "near field":***
    - ***Magnitude of cyclic stresses & number of loading cycles imposed by ground shaking.***
    - ***Number of loading cycles between the pile & soil.***
  - ***Pile installation method.***
  - ***Partial drainage and hence loading rate.***
  - ***Soil characteristics.***
  - ***Pile foundation stiffness [FEM results]***
- ***Differences in p-y behavior by different investigators can be largely explained by considering such factors.***

Slide 14

Summary Cont'd

- ***Complex cyclic p-y behavior of liquefying soil is only crudely approximated by simplified pushover analyses (limit equilibrium or beam on nonlinear Winkler foundation).***
  - ***The appropriate p-y parameters (pressures) used to envelop the response in a pushover analysis depend on the details of the design procedure.***
  - ***The importance of the p-y parameters for the liquefied soil depends on the loading mechanism (e.g., dominating load from crust?).***
  - ***1st order effects of D_R and loading condition (peak virgin cycle vs. numerous smaller cycles) may be the most important considerations for design.***
- ***Centrifuge data also being analyzed by BNWF pushover and dynamic FEM analyses to evaluate design guidelines.***