Pile response to liquefaction and lateral spreading: field observations and current Research

The Fifteenth Spencer J. Buchanan Lecture

By **RICARDO DOBRY**

Friday November 9, 2007
College Station Hilton
College Station, Texas, USA
http://ceprofs.tamu.edu/briaud/buchanan.htm

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SPENCER J. BUCHANAN

Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star.

He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.

Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England. They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy's Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.

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The College of Engineering and the Department of Civil Engineering gratefully recognize the generosity of the following individuals, corporations, foundations, and organizations for their part in helping to establish the Spencer J. Buchanan ’26 Professorship in Civil Engineering. Created in 1992 to honor a world leader in soil mechanics and foundation engineering, as well as a distinguished Texas A&M University professor, the Buchanan Professorship supports a wide range of enriched educational activities in civil and geotechnical engineering. In 2002, this professorship became the Spencer J. Buchanan ’26 Chair in Civil Engineering.

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AGENDA

The Fifteenth Spencer J. Buchanan Lecture
Friday November 9, 2007
College Station Hilton

2:00 p.m.	Welcome by Jean-Louis Briaud
2:05 p.m.	Introduction by David Rosowsky
2:10 p.m.	Introduction of Raymond Krizek by Jean-Louis Briaud
2:15 p.m.	"Dredged Materials: Friend or Foe?" The 2006 Terzaghi Lecture by Raymond Krizek
3:15 p.m.	Discussion
3:25 p.m.	Introduction of Ricardo Dobry by Jean-Louis Briaud
3:30 p.m.	"Pile response to liquefaction and lateral spreading: field observations and current research" The 2007 Buchanan Lecture by Richard Dobry
4:30 p.m.	Discussion
4:40 p.m.	Closure with Philip Buchanan
5:00 p.m.	Photos followed by a reception at the home of Jean-Louis and Janet Briaud.

Ricardo Dobry

Ricardo Dobry earned his B.S. from the University of Chile, his M.S. from the National University of Mexico, and his Sc.D. from the Massachusetts Institute of Technology, all in civil engineering. He has taught at MIT, U. of Chile, U. of Texas, Austin, and since 1977 has been a member of the faculty at Rensselaer Polytechnic Institute, where he currently serves as Director of the Center for Earthquake Engineering Research which includes the NEES RPI centrifuge experimental site. He holds the title of Institute Professor of Engineering with an endowed chair at the RPI level.

Dr. Dobry's research interests include soil dynamics, geotechnical earthquake engineering and geotechnical dynamic centrifuge testing. He was a participant of the group that wrote the new seismic provisions on local site amplification in the 1990's now incorporated in U.S. building codes. He is one of the authors of the 20-year research plan in earthquake engineering prepared in 2003 by the Earthquake Engineering Research Institute for the National Science Foundation (NSF) in the U.S. Since 2000 he has directed the Rensselaer geotechnical centrifuge experimental site of the Network for Earthquake Engineering Simulation (NEES), one of 15 electronically interconnected experimental nodes funded by NSF to revolutionize earthquake engineering research in the U.S. He served as a member of the first Board of Directors of NEES in 2003-04. He has written more than 200 technical papers and research reports and has directed 40 PhD and MS theses at Rensselaer.

Dobry has served as consultant and member of consulting boards of a number of civil engineering projects, including offshore oil platforms in Venezuela and Australia, earth dams and dikes in California, Puerto Rico and South America, seismic retrofitting of several large bridges in New York City (NYC), seismic guidelines for design of new bridges in NYC, and design of the new Rion-Antirion Bridge in Greece. The Rion-Antirion Bridge was named the 2005 Outstanding Civil Engineering Achievement by the American Society of Civil Engineers (ASCE). Dobry has been an invited state-of-the-art and keynote speaker at international meetings in the U.S., Mexico, South America, Europe, Japan and Australia. He earned the J. James Croes Medal of ASCE in 1985, and was elected member of the U.S. National Academy of Engineering in 2004 "for fundamental contributions to multiple aspects of geotechnical earthquake engineering."

PILE RESPONSE TO LATERAL SPREADING:
FIELD OBSERVATIONS AND CURRENT RESEARCH

The Fifteenth Buchanan Lecture

Presented by Ricardo Dobry

PILE RESPONSE TO LATERAL SPREADING: FIELD OBSERVATIONS AND CURRENT RESEARCH

R. Dobry, C. Medina, T. Abdoun, Rensselaer Polytechnic Institute, Troy, NY
S. Thevanayagam, University at Buffalo, Buffalo, NY

Introduction

The lecture focuses on the response of pile foundations to lateral spreading induced by liquefaction, and I will be discussing first case histories followed by a couple of research issues. A list of references covering both lateral spreading and its effect on piles is included at the end of the document.

I want to acknowledge first the people and organizations listed below that have been critical both in our research as well as in the preparation of this lecture. They include research partners in the U.S. and Japan, students and staff at three universities, and the Network for Earthquake Engineering Simulation and National Science Foundation in the U.S.A:

- Drs. Akio Abe, Ahmed Elgamal, Usama El Shamy, Lenart Gonzalez, Masayoshi Sato, Kohji Tokimatsu and Mourad Zeghal
- Graduate Students at RPI, U. at Buffalo, and U. of California San Diego
- Staff at U. at Buffalo-NEES and RPI-NEES
- Network for Earthquake Engineering Simulation (NEES)
- National Science Foundation

Lateral spreading due to liquefaction of saturated cohesionless soils is a common and extremely damaging phenomenon occurying during earthquakes. It typically happens near waterfronts or in mildly sloping areas, with permanent lateral displacements of the ground ranging from a few centimeters to several meters (e.g., Youd, 1993)

Figure 1 shows two photos of the damage caused by lateral spreading to piles 12 years ago in the Kobe earthquake in Japan. In general, liquefaction caused about 10 billions dollars damage in that earthquake and quite a bit of it was due to lateral spreading. Figure 1a shows a railroad bridge where the lateral displacement of the ground moved the base of the piers, causing the fall of a girder. Figure 1b shows the damage to a reinforced concrete pile foundation under a building.

(a) (b)

Figure 1. Damage caused by lateral spreading to piles during 1995 Kobe earthquake in Japan: (a) railroad bridge (M. Kumura, Kyoto U.); and (b) damage to reinforced concrete pile foundation (K. Tokimatsu, Tokyo Inst. of Technology).

In this lecture, I will be covering three subjects. First I will go over case histories, starting with a detailed discussion of a well documented case history in Japan and following with a number of comments on the state-of-the art. The second and third subjects are on recent important research results.

Case histories

Fukaehama Island Building, 1995 Kobe Earthquake

I want to cover in some detail an extremely interesting and very well documented analysis of a case history published by Miwa and his co-workers last year (Miwa et al., 2000, 2006). This and other Japanese groups have done an outstanding job in documenting and analyzing case histories of damage to pile foundations in the 1995 Kobe earthquake. This particular case history illustrates especially well the different phenomena that take place when the ground around a deep foundation liquefies and

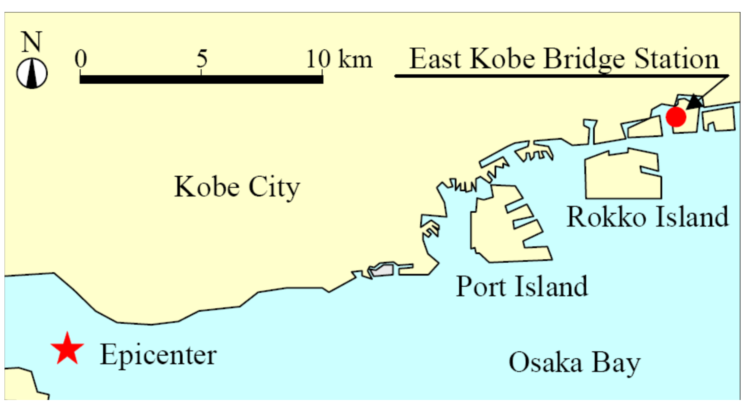


Figure 2. Locations of 1995 Kobe earthquake epicenter and investigation site (Miwa et al., 2000).

moves laterally. Figure 2 shows the map with the location of the site, which is near the East Kobe Bridge recording station, a little bit to the East of Port and Rokko Islands.

The structure in question is a long 2-story building, founded on steel pipe piles, where both the superstructure and foundation were extensively damaged by inertial effects, liquefaction and lateral spreading. Figure 3 shows the location of the building. There is another interesting case history of pile foundation damage due to liquefaction under another, 5-story building nearby, which was analyzed in detailed by Uzuoka and his group (Uzuoka et al., 2007). These two buildings are located in the Fukaehama artificial Island in Kobe, where there was extensive liquefaction throughout the island, and where there was also significant lateral spreading at locations near the quay walls. One extremely important aspect of the analyses of these two buildings is that they could take advantage of the records of the East Kobe Bridge Station, shown as a red circle on Fig. 3. This is a borehole array which produced good free field accelerograms of the earthquake at the ground surface and also at 30 m depth, under the liquefied layer.

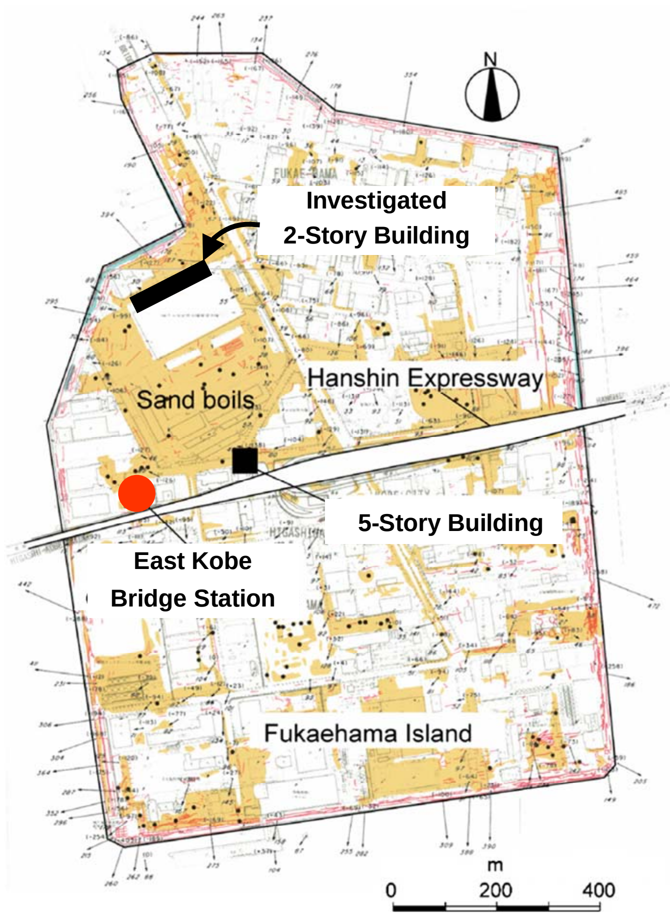


Figure 3. Location of investigated building (Uzuoka et al., 2007).

Figure 4a shows the plan view of the long 2-story structure, located not far from the quay wall so that it was affected by lateral spreading. The ground moved close to 1 m toward the quay walls near Piles SP1 and SP2, and about 50 cm near the piles more distant from the quay wall, like pile SP17, and because the piles were very flexible they displaced almost as much as the free field. This 2-story building had a steel frame structure, except for the columns of the first floor which were reinforced concrete. As shown in Fig. 4b, the foundations consisted of isolated footings supported by 2×2 or 2×3 pile groups made out of 400 mm outside diameter steel pipe piles nearly 30 m in length.

After the earthquake, it was found that several of the first floor reinforced concrete columns had failed in shear. The field investigation underground, using an inclinometer and a TV camera inserted into several of the pipe piles, showed that the piles had deformed to a significant depth, and also that the pile tops and concrete footings had cracked and deformed significantly.

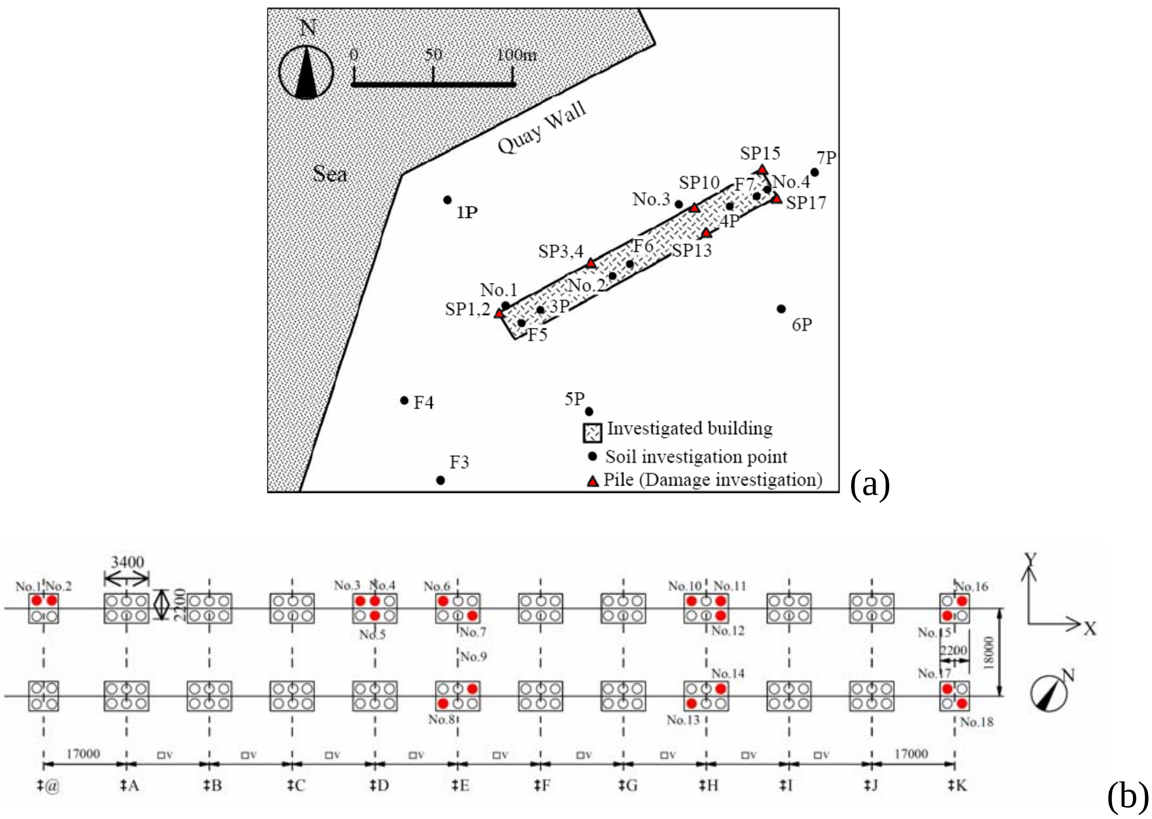


Figure 4. Plan view of the long 2-story structure investigated: (a) locations of building and soil investigation points; and (b) piles investigated (red dots) (Miwa et al., 2006).

Figure 5 presents the soil profile at the site, with the groundwater level at a depth of about 4 m, and a layer of artificial fill sand locally known as Masado extending from the ground surface to a depth of about 15 m. Just looking at Fig. 5 suggests that the Masado sand must have liquefied during the Kobe earthquake between a depth of about 6 m and the bottom of the layer at 14 or 15 m, because the sand in this range of depths has an SPT blowcount of 15 or less, and the shear wave velocities are also low. This is exactly what the investigation found: that the Masado layer between 6 m and 15 m liquefied and spread laterally, while the shallower part of the layer between the ground surface and 6 m depth acted as a nonliquefied soil crust riding on top of the deeper liquefied soil. The red lines show the deformed shapes of two of the piles, obtained from the inclinometer readings conducted as part of the investigation, and with the displacement of the tops of these two piles being somewhat less than 1 m (Fig. 5).

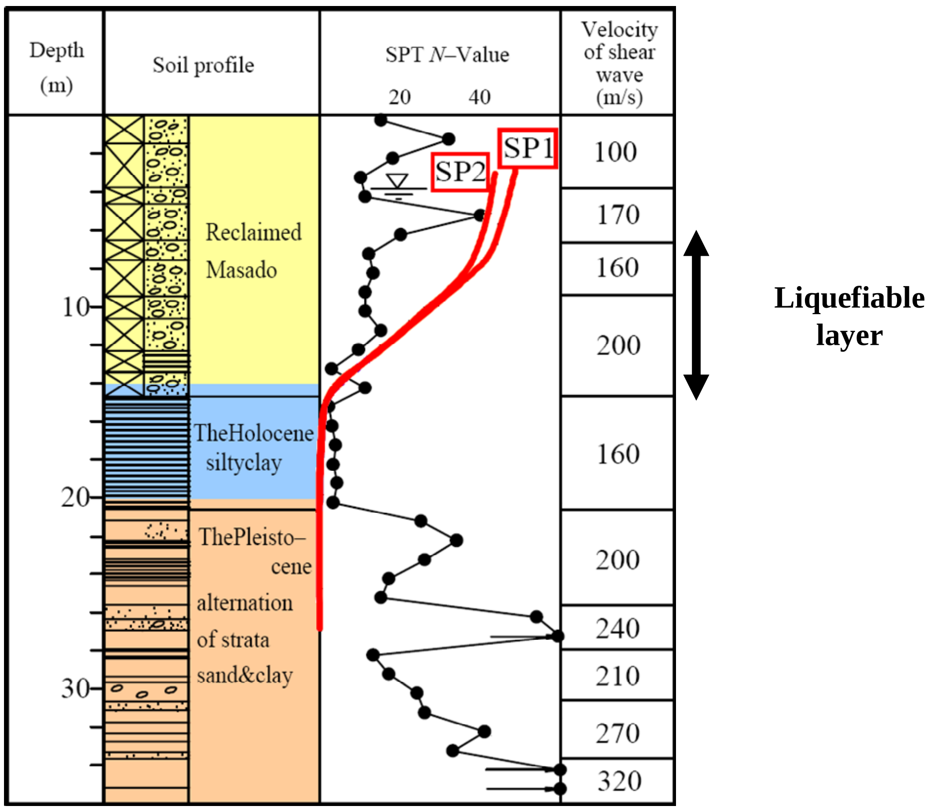


Figure 5. Soil profile at the investigated site, including SPT N-Values and shear wave velocities (Miwa et al., 2006).

The authors conducted a one-dimensional site response analysis using an effective stress program developed by Dr. Iai (Iai et al., 1992), to predict the acceleration and pore pressure responses of the soil in the free field. Figure 6 shows in red the calculated ground surface acceleration (Fig. 6a) and the excess pore pressure ratio calculated at the upper part of the liquefied layer (Fig. 6b), showing that liquefaction must have occurred at about 11 sec. Now, what lends this analysis an extraordinary degree of credibility, is that the base input motion used in the analysis was the acceleration recorded at 30 m depth at the borehole array nearby, and also that they were able to compare the computed ground surface acceleration with the recorded one, with excellent agreement as shown in Fig. 6a. Now, while the soil may have liquefied at 11 sec, some of the largest ground accelerations occurred 3 or 4 sec before liquefaction, at about 7 or 8 seconds.

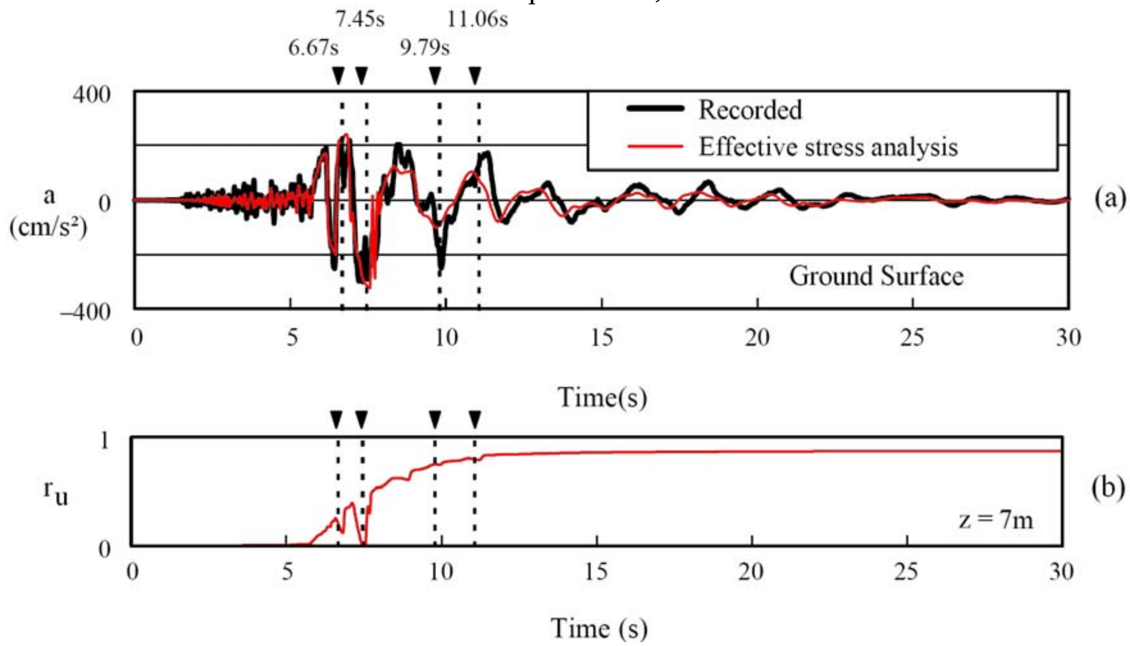


Figure 6. Time histories of: (a) acceleration at the surface; and (b) calculated excess pore water pressure ratio in liquefied layer (Miwa et al., 2006).

Then, as a final step in the calculations for this case history, the authors analyzed a simplified model of the structure consisting of one structural frame on top of two footings supported by the corresponding pile groups. The analytical model for this dynamic soil-pile-structure interaction calculation is shown on Fig. 7.

The foundation was subjected to the free field accelerations calculated before (Fig. 6a), which were the inputs to the pile and footings at different elevations. Without going into the details of what the authors did, it is clear that the near field horizontal response was modeled by nonlinear springs characterized by p-y curves, with these springs connecting the piles to the free field, and including softening of these springs as the Masado layer built up pore pressure and liquefied.

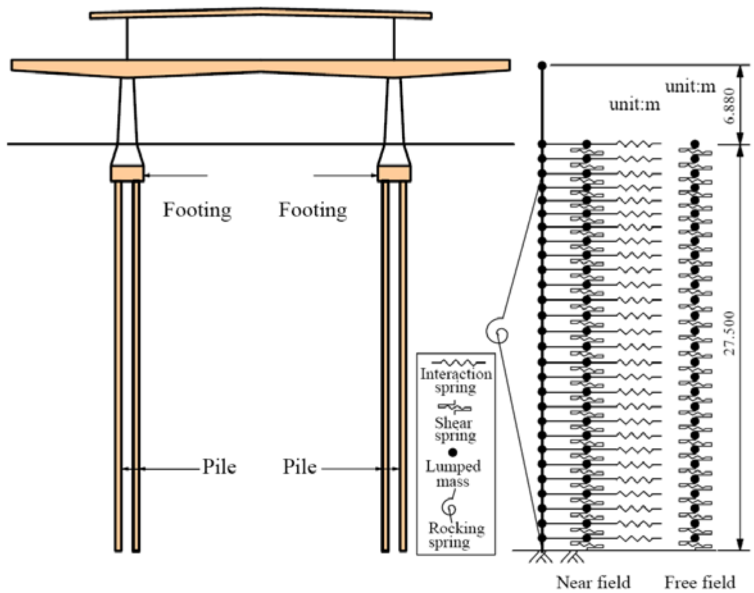


Figure 7. Analytical model for soil–pile-structure interaction (Miwa et al., 2006).

Figure 8c shows one of the results of this dynamic soil-structure interaction analysis, in this case the calculated pile bending moment at 14.75 m, which is near the base of the liquefied layer. The field accelerations (Fig. 8a) and pore pressures (Fig. 8b) are also shown. The authors reasonably concluded from this bottom plot that the piles must have exceeded the ultimate plastic bending moment at this depth at about 10 or 11 seconds, just at the time when the liquefaction process was being completed and the pore pressure ratio was reaching 100%, and that this pile failure was produced essentially by large cyclic oscillations of the ground, that is, it was a kinematic Soil-Structure Interaction (SSI) phenomenon with little or no influence of the inertia of the superstructure. On the other hand, the results of the same analysis for the structure itself, and for the footings and pile connections, suggested that the failure of the first floor concrete columns and of the pile tops must have occurred at about 7 or 8 seconds, fully 3 seconds before liquefaction, and that these failures were purely due to inertial loading, as the pore pressures in the ground at that time were still low.

Figure 9c shows a profile of the calculated pile bending moment at 9.79 sec, that is at about the time of liquefaction of the Masado layer (Fig. 9a), which is also the time of maximum cyclic kinematic effect on the pile bending moments. This shows again that, indeed, the ultimate plastic bending moment of the piles was exceeded at this time at about 15 m, and it was also exceeded simultaneously at a depth of about 8 m, which is near the top of the liquefied layer (Fig. 9c). In other words, the analysis predicts damage to the piles at those two depths, in accordance with the observations, because of large kinematic cyclic bending moments at the top and bottom of the liquefiable layer in the presence of a nonliquefiable soil crust, which is a typical pattern of damage observed in a number of case histories, and which has also been observed in centrifuge and 1g shaking tests. Figure 9 also shows the maximum cyclic ground shear strain calculated in the free

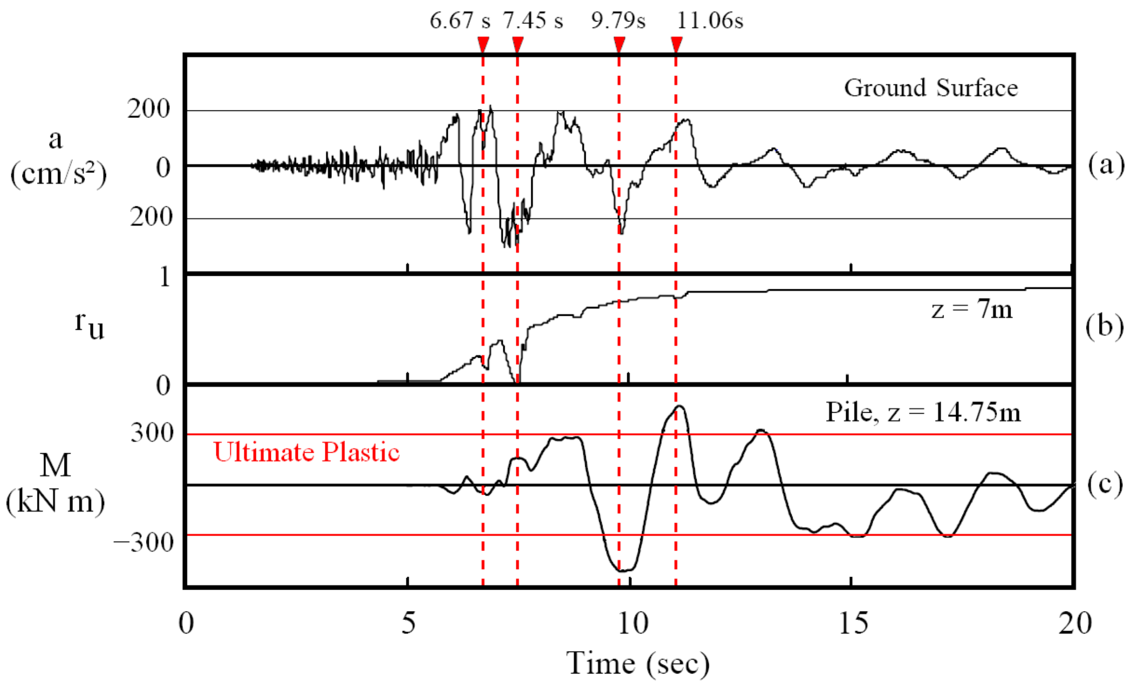


Figure 8. Time histories of the responses of piles and soil: (a) acceleration at the ground surface; (b) excess pore water pressure ratio in liquefied layer; and (c) bending moment of pile at z = 14.75 m (Miwa et al., 2006).

field (Fig. 9b), and as the curvature is the derivative of the strain, this shows clearly that the ground is trying to impose on the piles maximum curvatures at these same depths of about 8 and 15 m, which explains why the pile bending moments are maximum at those depths.

This dynamic analysis does not say anything about the additional effect caused by lateral spreading of the ground on the piles, which was a purely kinematic and static effect and which definitely occurred after liquefaction. Lateral spreading was evaluated separately by the authors, who concluded that the maximum bending moments and damage to the piles due to lateral spreading were again concentrated at the top and bottom of the liquefied layer, reinforcing the damage started by the cyclic effects during shaking at those same locations along the piles.

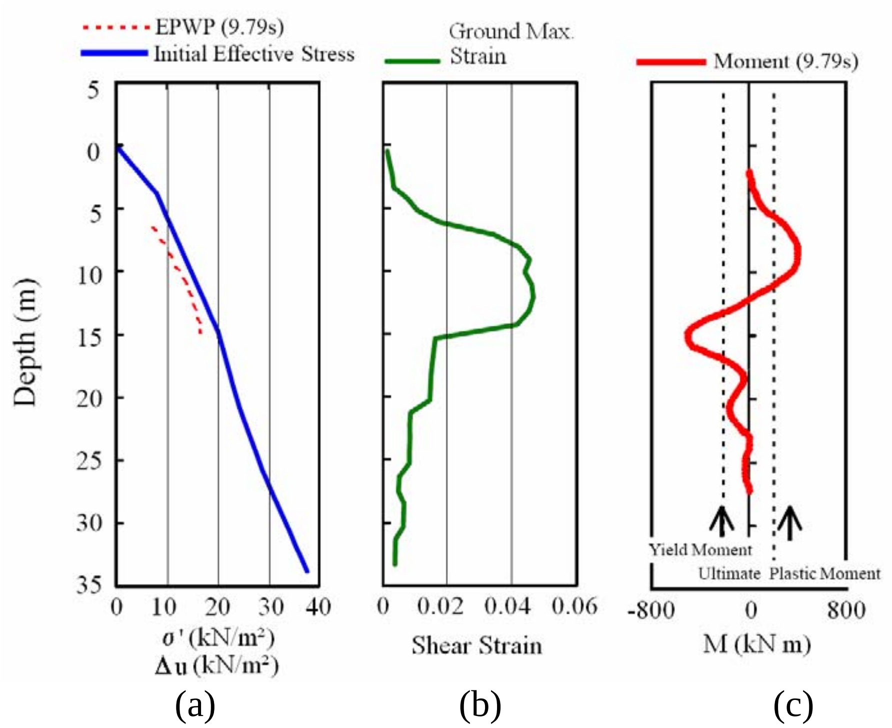


Figure 9. Response of soil and piles computed by dynamic soil–pile-structure analysis: (a) excess pore water pressure; (b) maximum cyclic ground shear strain; and (c) bending moment in the pile (Miwa et al., 2000, 2006).

Some Lessons from this Case History

- *Inertial* damage to superstructure and pile at *shallow* depths may occur much before liquefaction.
- *Kinematic* damage due to large *cyclic* ground deformations associated with *liquefaction* tends to concentrate at the *two interfaces* between liquefied and nonliquefied layers; it occurs just before or after soil liquefies.
- *Kinematic* damage due to large *permanent* ground deformation associated with *lateral spreading* also tends to concentrate at the top and bottom of liquefied layer; it develops after soil liquefies.

In summary, this case history illustrates very well, and in this case in an unusually clear time sequence, one loading after another, the three main causes of damage to pile foundations when there is shaking, liquefaction and lateral spreading. I must caution here that the fact that the inertial damage happened so long before liquefaction seems to be a particular feature of this example, and there are certainly other case histories where the inertial and kinematic cyclic effects have occurred more closely bunched together in time, shortly before or after liquefaction.

Summaries of Case Histories

I am listing below some main publications containing summaries and discussions of case histories of liquefaction and lateral spreading during earthquakes and their effects on deep foundations. The list of references at the end of this document provide the necessary publication details.

- NCEER Case Studies of Japanese and U.S. Earthquakes (Hamada and O'Rourke, 1992; O'Rourke and Hamada, 1992)
- Two Special Issues on Kobe Earthquake of 'Soils and Foundations' Journal, 1996 and 1998
- Tokimatsu (1999)
- Dobry and Abdoun (2001)
- Oregon State University Report (Dickenson et al., 2002)
- Ishihara (2003)
- U. California Davis Report (Boulanger et al., 2003)
- Bhattacharya et al. (2004)
- Proc. Workshop U. California Davis (Boulanger and Tokimatsu, 2006)

Some General Lessons from Case Histories

The following are some of the general lessons that have emerged from field observations and case histories of piles subjected to lateral spreading:

- Free field surface ground deformation D_H is a very important parameter.
- The spatial variation of D_H under the structure may contribute to damage.
- Top of piles may move about same as D_H, or much less if very stiff foundation (high EI, pile groups, batter piles, superstructural constraints).
- If there is a shallow nonliquefied crust above liquefied layer, passive thrust of that crust is a key factor.
- Damaging maximum bending moments tend to occur at top/bottom boundaries of liquefied layer.

Some Unsolved Engineering Questions

There are a number of important unsolved engineering questions on how to approach the analysis and design of deep foundations subjected to liquefaction and lateral spreading.

- How to combine inertial, cyclic kinematic, and lateral spreading loadings acting on foundation.
- Use of sophisticated (FEM) and simplified engineering analyses (p-y, pushover / limit equilibrium) for evaluation of lateral spreading effects on piles.

- Can lateral pressure of liquefied soil be neglected compared with passive thrust of nonliquefied crust?
- Effect of liquefied soil permeability.

An important question here is how to combine the three loadings: inertial, cyclic kinematic, and pseudostatic lateral spreading loadings. Also, there are a number of evaluation methods used in practice for evaluating response to lateral spreading, ranging from sophisticated FEM analyses to p-y curve or nonlinear spring analyses, and there are limit equilibrium methods that in their simplest form reduce to a static pushover analysis. When to use one or the other of these methods, and, especially, the details on how to use them, are still not clear, especially when the properties of the liquefied soil or of the soil in process of liquefying are important to the response of the piles.

Ongoing Research and Research Tools

- In-depth studies of case histories (mostly in Japan) using advanced technologies.
- Field tests with blasting (Rollins et al., 2005; Ashford et al., 2006).
- Large-scale 1 g shaking tests in Japan and U.S. (6m tall inclined laminar boxes), use of advanced sensors.
- Small-scale centrifuge testing.
- Use of advanced Information Technology tools for data integration, system identification and visualizations.
- Numerical simulations and analyses (Discrete Element Method (DEM), FEM, dynamic and static p-y, limit equilibrium / pushover analyses). .

Centrifuge and 1g modeling of pile foundation in liquefiable soil that reaches ground surface (p-y curves of liquefied soil)

Let me know address the first of the two research subjects promised at the beginning of the lecture. This is the character and quantification of the interaction between pile and liquefied soil that takes place after liquefaction, with or without lateral spreading. This is very usually done through the use of p-y curves and derived force-deformation loops, which are really nonlinear horizontal springs linking the pile or pile group to the liquefied soil in the free field. The issue of what are the p-y curves of the soil already liquefied is especially important in cases where the liquefiable layer reaches the ground surface, like for example in many bridge foundations.

Figure 10 shows the analytical model used by Miwa and his co-workers (Miwa et al., 2000, 2006) for the evaluation of the steel pipe piles in the case history I discussed before for the two-story building, when subjected to lateral spreading in the free field after liquefaction. It is a static p-y model, clearly with very soft nonlinear springs between 6 and 15 m where the soil liquefied, and with the deformation of the pile being driven by the soil crust shallower than 6 m which did not liquefy. In this case the exact number and character of the springs representing the liquefied soil below 6 m is not very important.

But in other cases when the whole layer liquefies up to the ground surface, then the p-y curves of the liquefied curve are the only game in town, and they completely control the deformation of the pile and the corresponding bending moments.

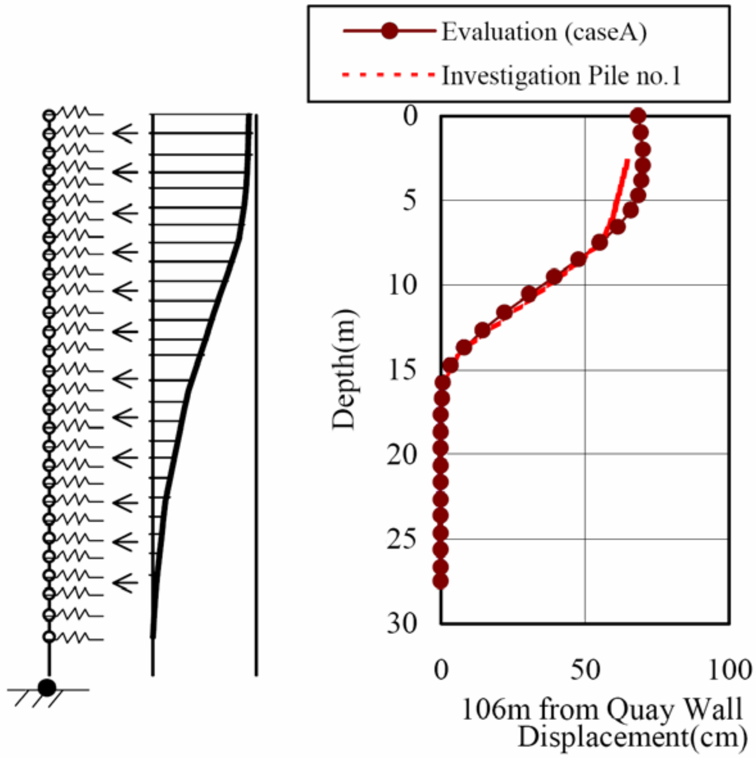


Figure 10. Residual displacement due to lateral spreading of the ground evaluated by analytical model used by Miwa (Miwa et al. 2000, 2006).

One thing we know about p-y curves of liquefied sand is that they tend to be strongly nonlinear, with very low values of resistance p at small relative displacements, y, and with the resistance increasing violently at larger displacements due to the dilative response and negative induced pore pressures close to the pile in what is often close to an undrained shear loading situation. Figure 11 shows instantaneous p-y response of the liquefied soil during shaking, calculated through System Identification from centrifuge results at the U. of California at Davis by Dr. Wilson and his co-authors (Wilson et al., 2000). There is now quite a body of evidence which shows this type of nonlinearity due to dilative response, including the centrifuge measurements in Fig. 11, as well as full scale shaking table tests, and more recently field pile loading tests in soil that had been liquefied by blasting.

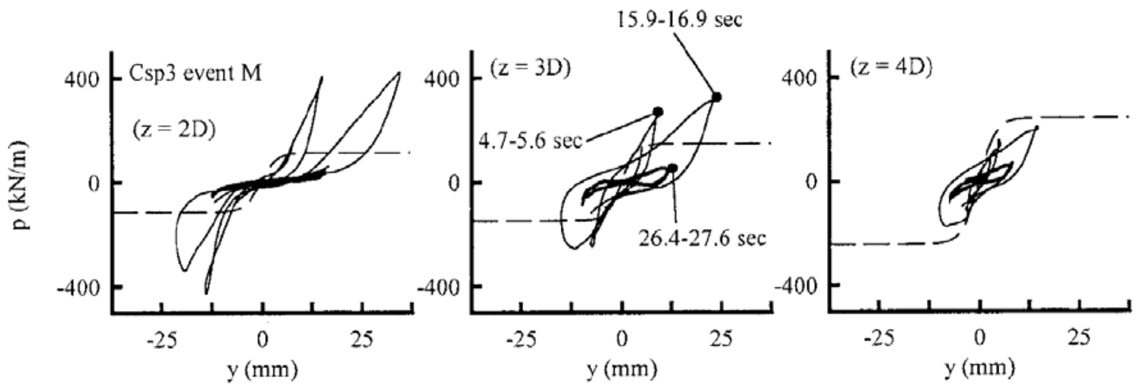


Figure 11. Instantaneous p-y response of the liquefied soil during shaking, calculated using System Identification from centrifuge results at the U. of California at Davis (Wilson et al., 2000).

Other researchers have also found significant experimental evidence that the lateral resistance between pile and soil, that is the value of p, increases when the rate of loading increases, and have proposed replacing the p-y curves by relations between p and the relative velocity between the pile and the soil, V = dy/dt, that is they would replace the spring by a dashpot, typically a linear dashpot. This would mean modeling the liquefied soil as a viscous fluid rather than as a solid or semisolid having a well defined shear strength. Both Newtonian and Non-Newtonian formulations with constant material viscosity, or with viscosity decreasing with strain rate, have been proposed (e.g., Hwang et al., 2006).

Figure 12 shows one of the proposed viscous models, in this case a Non-Newtonian model recently published by Hwang et al. (2006), where the viscosity of the liquefied soil decreases as the strain rate increases. One main problem of using linear models like this in practical pile-soil interaction analysis is that they ignore the strong evidence of nonlinearity. But on the other hand, the evidence behind these viscous models raises a legitimate question on how to put together a consistent picture for the lateral interaction between pile and liquefied soil, which takes into account both effects of nonlinearity and velocity of loading.

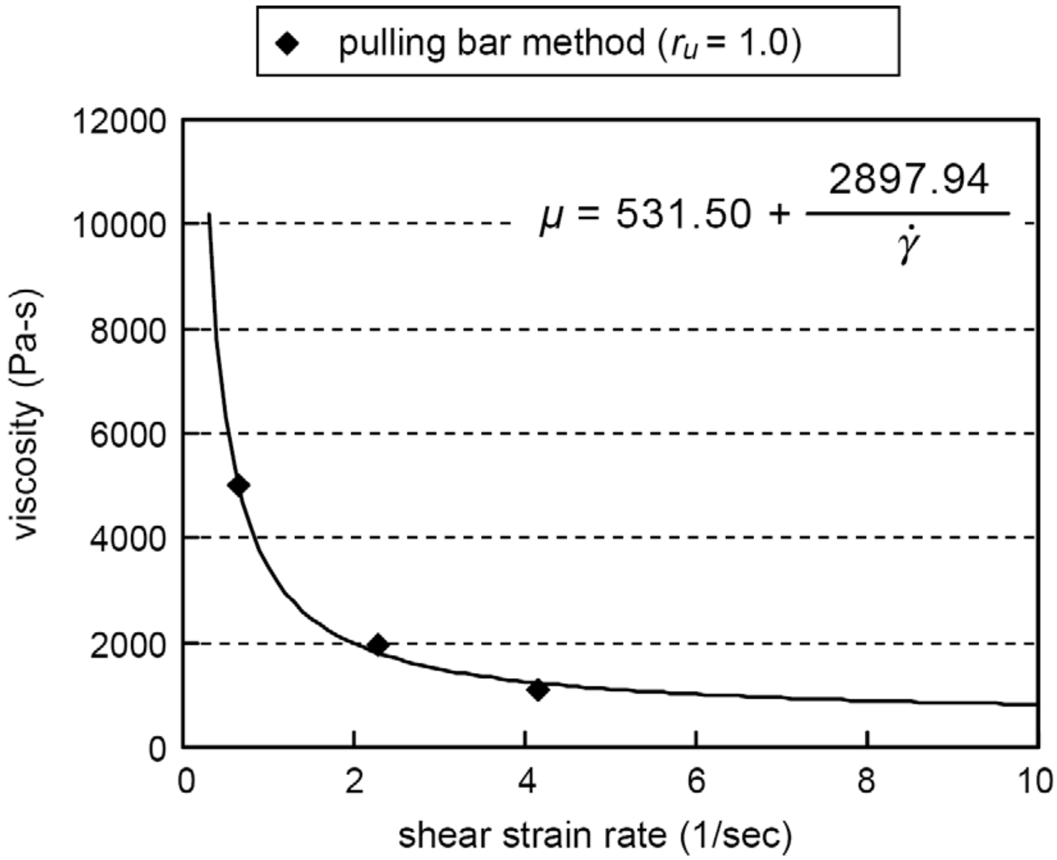


Figure 12. Liquefied soil modeled as a Non-Newtonian viscous fluid (Hwang et al., 2006).

A recent paper by Dungca and co-workers (Dungca et al., 2006) shows how this can be done (Fig. 13). In these experiments where the pile was represented by a buried cylinder moving back and forth through the liquefied soil, the effect of loading rate was measured, and is represented in the plot by the relative velocity between the free field and the moving cylinder they used to represent the pile. This relative velocity, V, ranged from 1 mm/sec to 100 mm/sec in the experiments. The authors found that, indeed, as the velocity V decreases from 100 to 1, the maximum value of the force p in the measured nonlinear p-y curves also decreases, which in principle would be consistent with a viscous model. However, as you can see the p-y loops again are strongly nonlinear in this figure, and the authors, instead of reaching for a viscous model, concluded that the effect of this decreasing loading velocity was due to the water flowing from the free field having more time to dissipate the negative pore pressures in the sand near the pile, therefore decreasing the effect of dilatancy which controls the observed nonlinearity. A number of other researchers including myself, strongly agree with this conclusion, which has also been suggested over the years by other investigations. That is, instead of modeling the liquefied soil as a simple viscous fluid, what is needed to predict the p-y curves of the liquefied soil is a basic effective stress soil mechanics model that captures the dilatancy effect in a partially drained situation, including the role of parameters such as soil permeability.

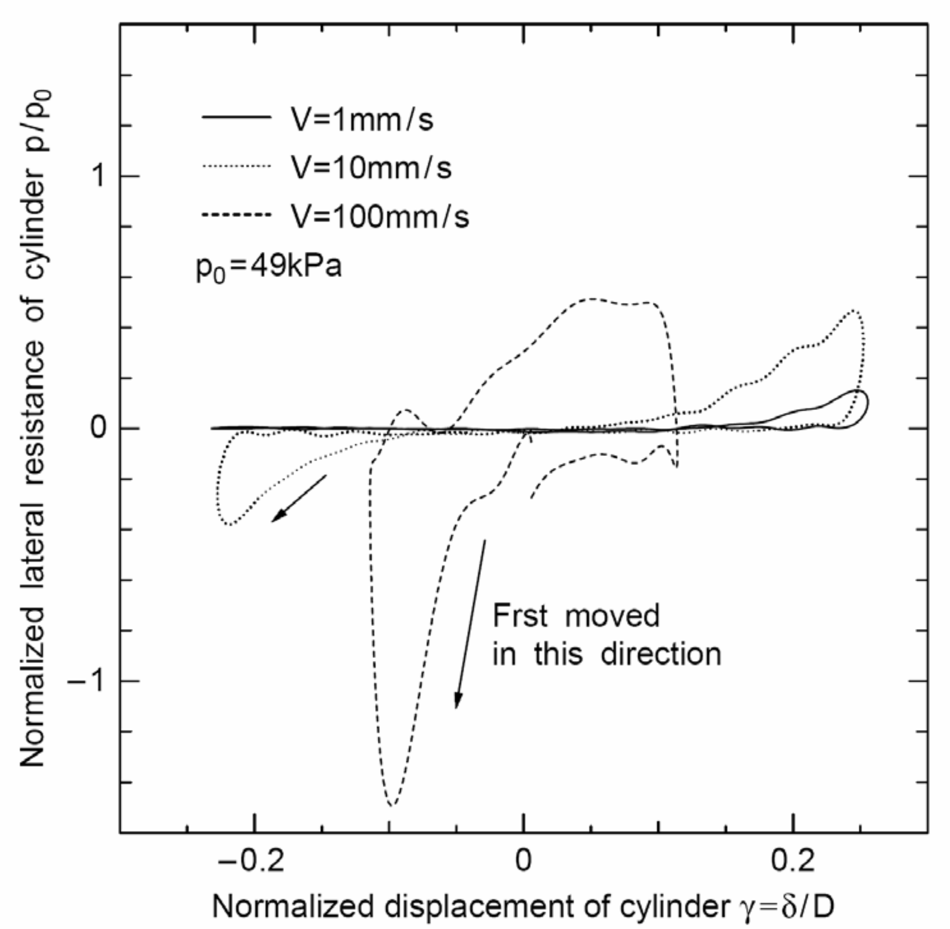


Figure 13. Effect of rate of loading on p-y curves of liquefied soil (Dungca et al., 2006).

Some General Lessons from Research on p-y Curves of Liquefied Soil

- p-y curves are strongly nonlinear, especially at large y, due to dilatant undrained stress-strain soil response near pile.
- Stiffness/strength of liquefied soil decreases when V = dy/dt decreases, because there is more time for water flow to dissipate negative pore pressures.
- Therefore, soil permeability is a key parameter determining apparently "viscous" p-y behavior.
- Possible corollary: more impervious liquefied soil should have stiffer p-y curves (silty sand stiffer/stronger than clean sand?).

Along these lines, I want to show you results of two centrifuge tests we conducted at RPI on the response to lateral spreading of a 2×2 pile group, which shows a big effect of soil permeability for a case where the liquefied soil layer does reach the ground surface. Figure 14 shows the basic setup we use at RPI for lateral spreading experiments

(Gonzalez et al., 2007). We have this rectangular laminar box container, which is made of a number of rectangular rings separated by roller bearings providing a very low friction contact; thus achieving the desired flexible wall conditions. We incline the box a couple of degrees to the horizontal to induce the desired lateral spreading. At 50 g, the 16-cm pile embedded in the loose saturated sand layer represents a prototype end-bearing pile 8 m in length. We subject the base of the box to shaking that liquefies the loose sand and induces lateral spreading, with the liquefied soil then pushing against the piles and pile cap as it happens in the field. We conducted two tests with this setup, with the two tests being identical in all respects, except that in one we used water as the pore fluid representing a prototype coarse sand of high permeability, and in the other test we used a viscous pore fluid 50 times more viscous than water, to represent a prototype fine sand having a permeability 50 times lower. Both tests induced a similar lateral spreading of about 1.5 m in the free field, but the response of the pile group was radically different.

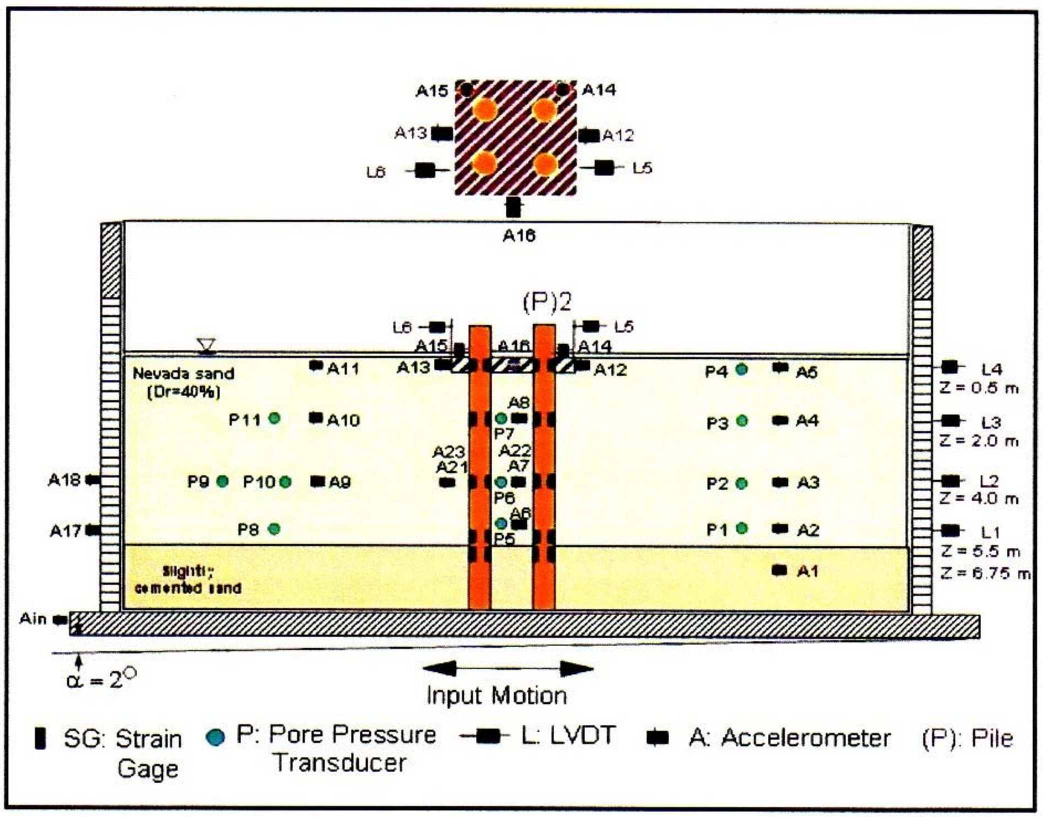


Figure 14. Setup of two centrifuge tests using water and a viscous pore fluid to simulate different permabilities (Gonzalez et al., 2007)

Figure 15 presents the measured pile bending moment at the bottom of the liquefied layer, plotted versus the measured displacement of the top of the pile group. The red line corresponds to the test of high soil permeability using water as pore fluid, while the blue line corresponds to the second test with a viscous pore fluid and thus low soil permeability. Because the pile group itself is a linear system, we expect these lines to be more or less straight lines, and in fact to follow the same general straight line path in both tests, which they do. But here is where the similarities end between the two tests. In the test using water, the piles developed a maximum bending moment of about 50 kN-m and a maximum displacement of 7 cm at some time during the shaking, bouncing back and decreasing the bending moment and the displacement after that. On the other hand, the piles in the other test developed a bending moment close to 400 kN-m and a displacement of 45 cm, without ever bouncing back. That is, in the test with low soil permeability, the dilatancy effect and corresponding negative pore pressures near the piles were strong enough, and exerted such a strong grip on the pile group, that the maximum pile group displacement and bending moment were increased by a factor of seven compared to the other test. This is quite dramatic, and while we do not quite believe that this ratio of seven can be directly extrapolated to the field because the applicability of some of the scaling laws is not completely clear in this case, there is also some evidence from full-scale 1g tests which suggest that this effect is still present in the field but to a smaller degree. In the next 1-2 years, we expect to learn much more about this from some large scale 1g pile tests scheduled under a NEES project (Dobry et al., 2007).

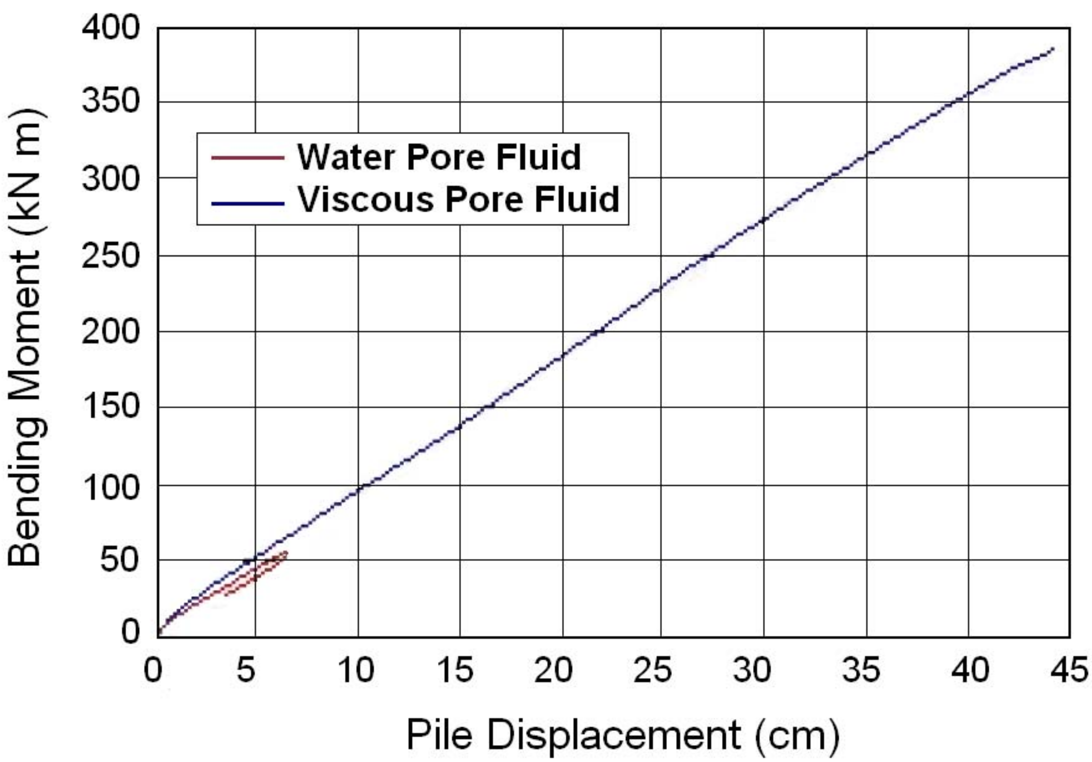


Figure 15. Influence of liquefied soil permeability on pile bending moment (Gonzalez et al., 2007)

Recent research on lateral spreading

As the last point of the presentation, I want to share with you some fascinating results we got very recently (May 2007), in a full scale test of lateral spreading – no piles, only free field. This test was conducted at the University at Buffalo under the direction of Prof. Thevanayagam, as part of a NEES collaboration (Dobry et al., 2007), and as far as I can tell they provide the first direct confirmation on how lateral spreading starts in a mild slope of homogeneous loose sand.

Figure 16 shows the 6-m tall laminar box container on the floor of the laboratory just before the test, inclined 2 degrees to simulate a mild infinite slope, with the box already filled with saturated uniform fine sand, ready to be shaken horizontally at the base with actuators connected to a strong wall. The sides of this box container are formed by aluminum rings separated by ball bearings to minimize friction. The sand was deposited wet in the box as a hydraulic fill at a relative density of about 55%.

Figure 16. 2-degree inclined laminar box before Test SG-1.

Figure 17 presents a plan view of the top of the box illustrating the extensive instrumentation placed inside the soil as well as on the rings outside the box. This included accelerometers in the soil and on the rings, piezometers in the soil, displacement transducers called potentiometers which were placed on the rings, and a new type of MEMS vertical array inside the soil developed by Prof. Abdoun, which measures time histories of acceleration and displacement in three directions and is labeled "Shape-Acceleration Array" (Abdoun et al., 2006, 2007). We have now a couple of hundreds of recorded time histories to play with from this test, and this abundance of data allows cross-comparisons of records checking different technologies against each other, and also verifying that the soil and the rings moved more or less together.

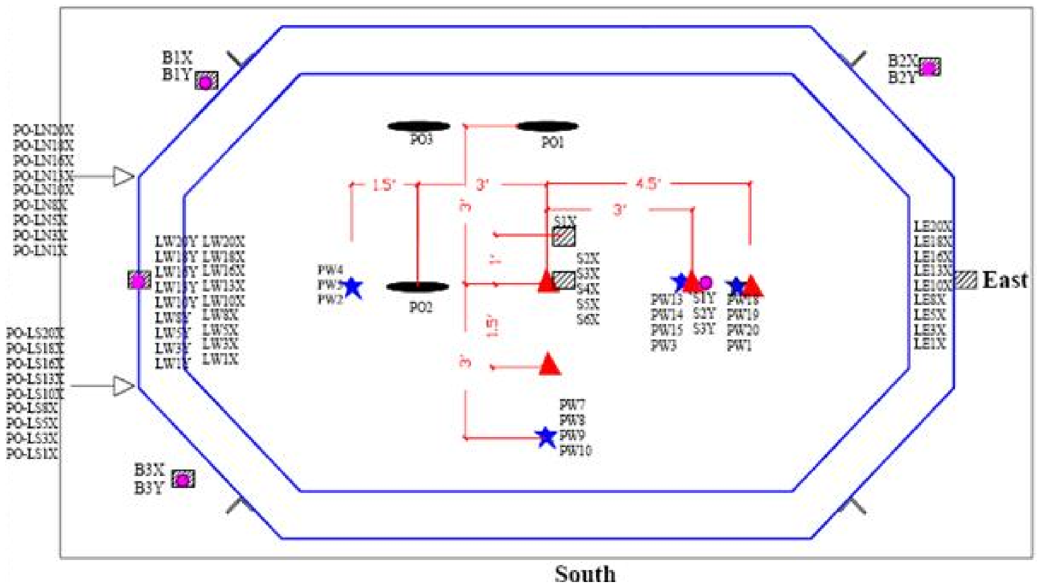


Figure 17. Plan view of the top of the box and sensor location.

Figure 18 shows the target input shaking. It included first 10 cycles of 0.01g shaking which was almost nondestructive, labeled "a_1" on the figure, followed by three sequences of 20 cycles of 0.05 g, 0.15 g, and 0.30 g, which are labeled a_2, a_3 and a_4. In the actual test the soil liquefied and developed 30 cm of lateral spreading in the first few cycles of 0.05 g, and the test was stopped after about half of the 0.05 g sequence, after only 8 sec of shaking. That is, we have data for a total of 8 seconds of shaking (record between 2.5 and 10.5 sec), but as you will see in the next few slides those 8 seconds of data contain a golden treasure of information.

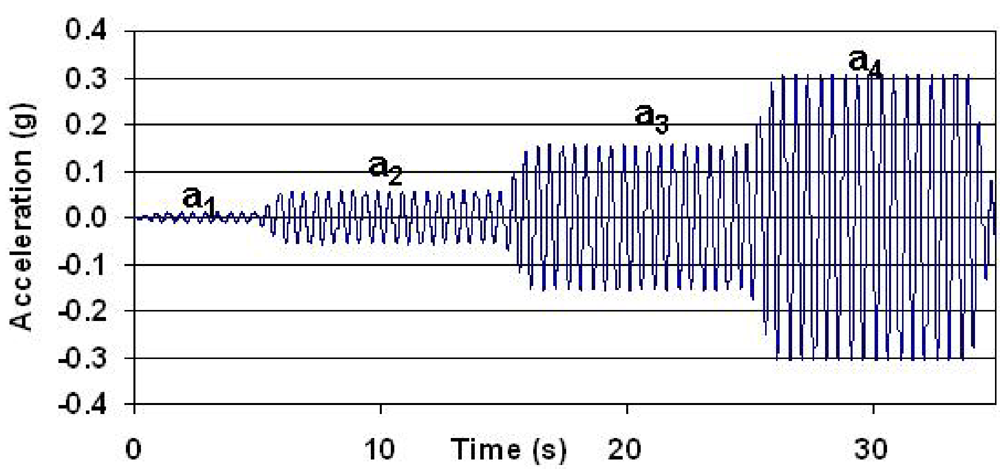


Figure 18. Input base excitation (acceleration of base).

First let me come back to the consistency of the different records, which was excellent in all cases. Figure 19 shows a comparison of lateral acceleration time histories measured at a depth of 3 m using a regular accelerometer placed on the corresponding ring at that depth, and by the Shape-Acceleration Array at that same depth but within the soil. You can see that the agreement between the two recordings is very good. The base shaking in this plot starts at 2.5 seconds, and here you can see clearly the 8 seconds of data.

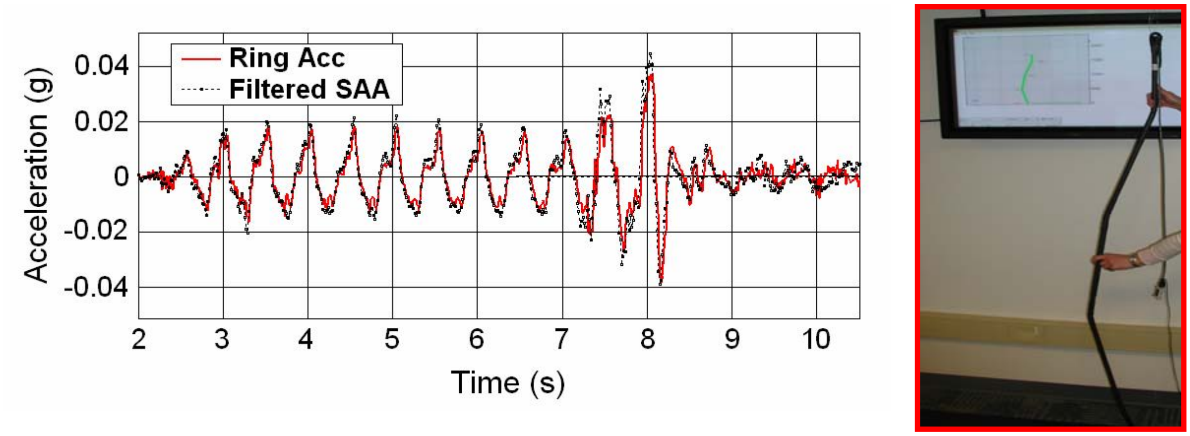


Figure 19. SAA vs Ring Accelerometer; 3m Depth.

Figure 20 shows a comparison between permanent lateral displacement time histories measured at the ground surface. As there was no displacement accumulation during the first 4 seconds of shaking, I am showing here the comparison only for the last 4 seconds. One of the two recording instruments is a potentiometer placed on the corresponding ring just below the ground surface, and the other recording instrument is the Shape-Acceleration Array sensing point at the ground surface. Again, the comparison is very good at all times, with a measured total lateral spreading at the end of about 30 cm.

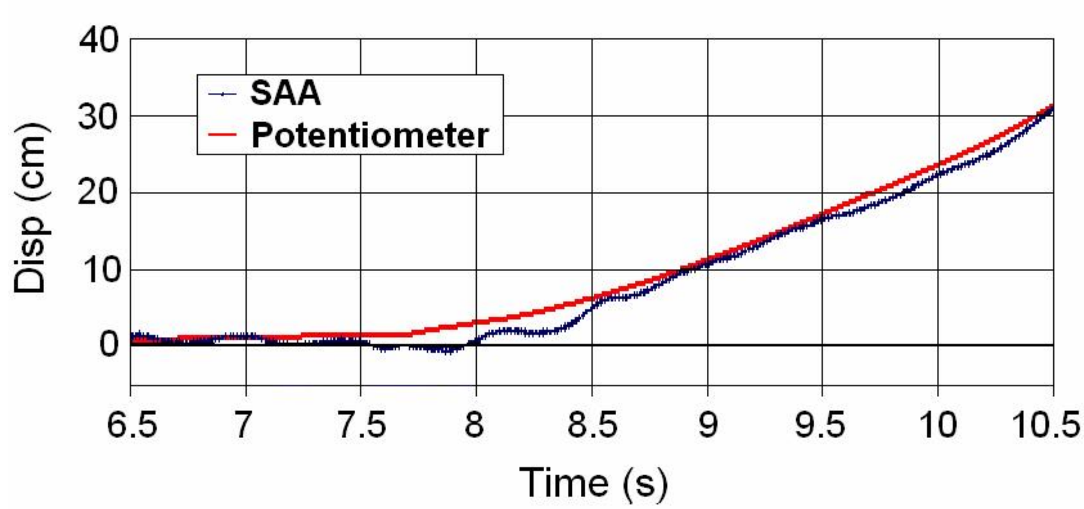


Figure 20. SAA vs Potentiometer Displacement at Ground Surface.

Figure 21 shows again the recorded 8 seconds of base excitation, but now in terms of lateral displacement of the base instead of base acceleration. This way of characterizing the base motion has some advantages for our purpose of looking into the mechanics of lateral spreading. Please pay attention to the fact that there are three distinct stages in these 8 seconds of shaking. First we have the very weak input shaking until about 7 sec, which as you will see did generate some pore pressures in the soil but essentially no lateral spreading. Then you have two transition cycles between 7 and 8 sec, which accelerated the pore pressure buildup and where the lateral spreading actually started, and finally we have the stronger cycles of shaking between 8 and 10.5 seconds which liquefied most of the layer and continued the lateral spreading accumulation.

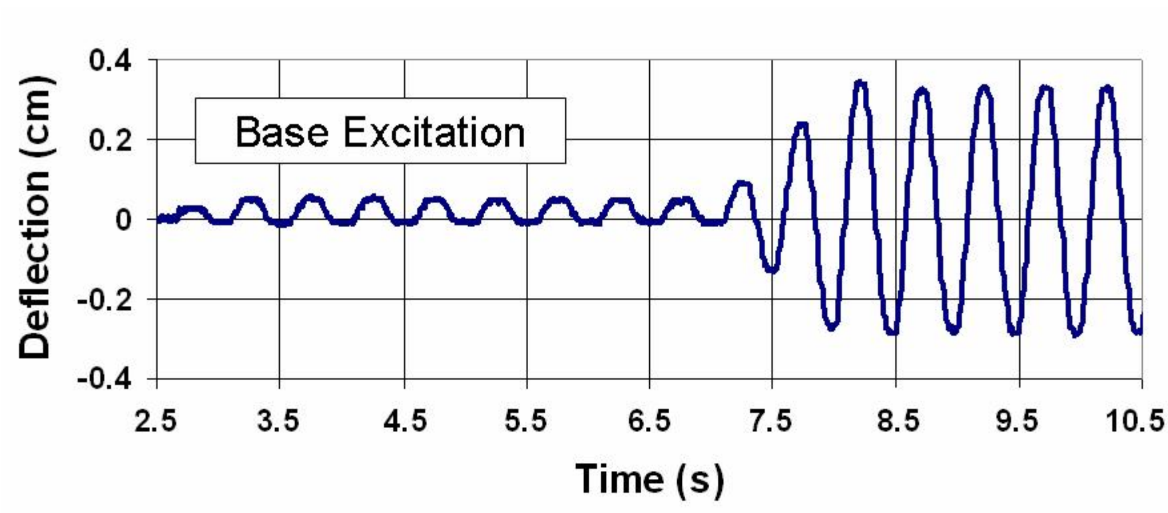


Figure 21. Input base excitation (lateral displacement of base).

Figure 22 presents the information from the piezometers, which confirms that there was some pore pressure buildup at the end of the weak shaking at 7 sec, and also that by the end of shaking essentially the whole sand layer had liquefied.

Figure 23 shows the promised gold. These are all the lateral displacement time histories measured by the potentiometers on the rings at the various depths labeled on the figure: at the ground surface, at 0.24 m depth, etc., until we have the record at 4.91 m depth, that is very close to the base of the stratum. The curve at the bottom is the same input shaking in terms of a displacement time history shown before. In this graph, positive deflection means the downslope direction while the negative deflections point upslope.

Let us look at this figure in some detail. During the weak shaking ending at 7 seconds, a small deflection does accumulate cycle after cycle, which at 7 sec is of the order of 1 cm, but with the whole soil layer essentially following the cyclic base motion. But after 7 seconds, when the first transition cycle occurs with larger amplitude of base shaking, something different happens. The top five or six curves, down to about a depth of 2 m "take off," that is they stop following the base and start accumulating rapidly permanent deformation in the downslope direction. This means that a shallow block of soil down to 2 m depth or so, starts sliding downslope while the rest of the soil layer below 2 m continues following the base. And then, in the next 1 or 2 cycles, that is somewhere between 8 and 8.5 sec, the rest of the soil layer also takes off and starts sliding essentially at the base at the stratum, with this rapid displacement accumulation at all depths continuing until the end of shaking.

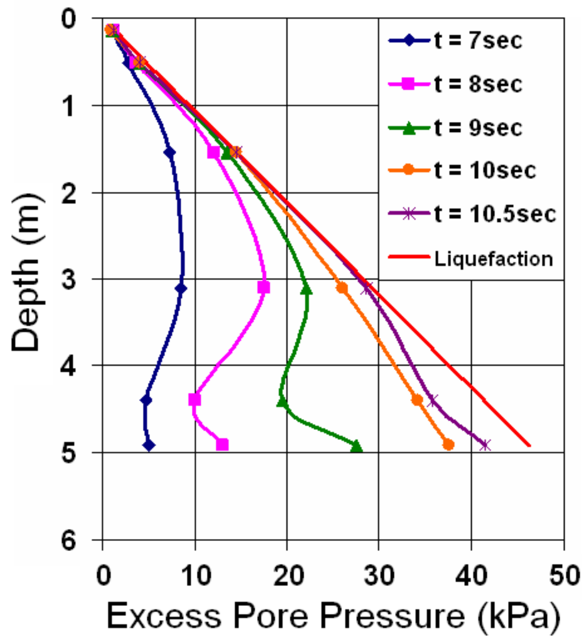


Figure 22. Pore pressure development.

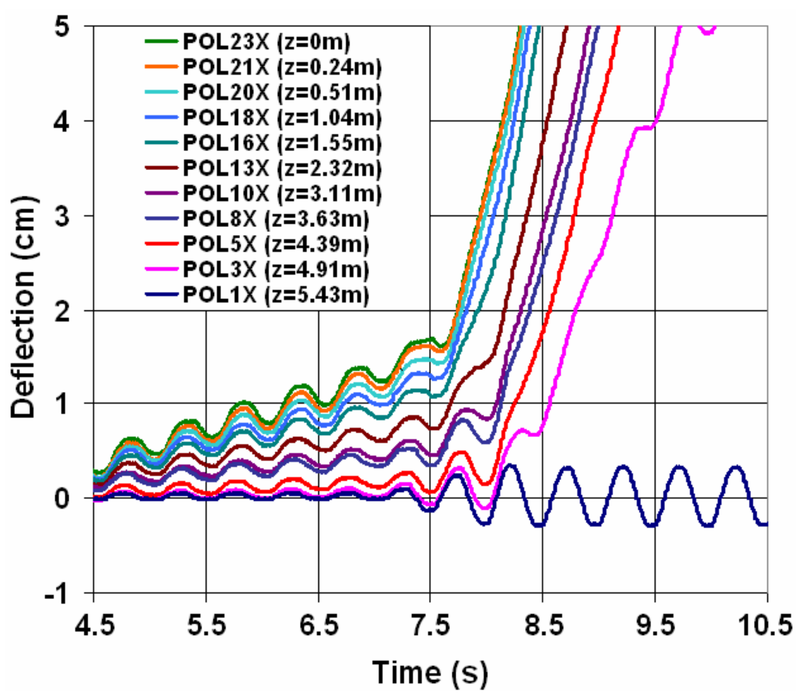


Figure 23. Lateral Spread Initiation: Top soil block slides at 7.5 sec at a depth of 1.5-3 m; and later a second soil block slides at 8-8.5 sec at a depth of 4.5-5.5 m, that is at the base of the soil layer.

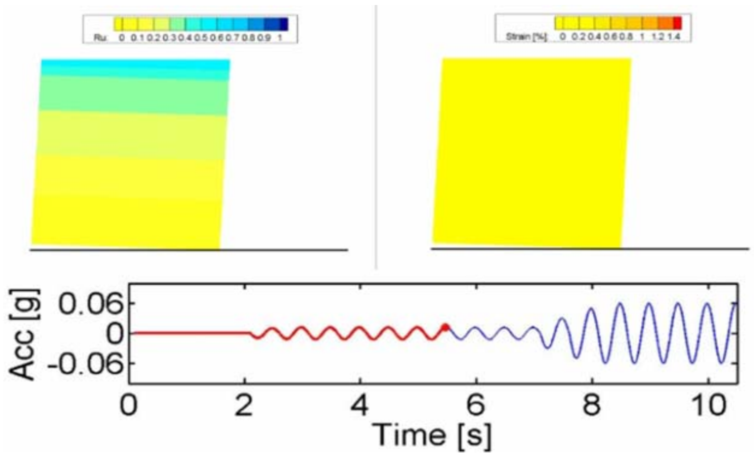
It is extremely interesting that each of these two block sliding events, starting near the top of the soil layer and continuing later at a deeper elevation, occurs at a specific part of the cycle of the base motion. Take the first event where a block of soil slid at 7.5 sec. This seems to happen when the base motion was at its maximum amplitude in the downslope direction, and started coming back upslope, with the soil trying to follow the base but failing to do so and instead continuing to move downslope. This fact, and also the fact that all lines of downslope deflection versus time after sliding are more or less parallel to each other, indicating close to a rigid body movement of the corresponding soil mass, strongly suggests that this may have the character of a rigid block "Newmark-type sliding." Also of great interest is the fact that the two failure surfaces or zones that formed successively, first at 2 m and then at the base of the soil, developed at a time when the soil at that elevation was not fully liquefied but was softer because it had built significant pore pressures, with pore pressure ratios ranging between 50 and 80%. Clearly we will be investigating and modeling this data for some time in the future, as in my opinion there is still quite a bit of gold to be dug out of this test.

Finally, Fig. 24 shows 4 snapshots of a visualization of this test, based on the records you just saw. Each frame in Fig. 24 has three parts. At the bottom you have the input acceleration time history, with the red dot indicating where we are in the shaking. The rectangle to the left shows the evolution of the pore pressure buildup, with dark blue indicating 100% pore pressure ratio. The rectangle to the right shows the development of horizontal shear strains in the soil, with a strong red color indicating that the accumulated strain has reached a value of at least 1.4% and where soil block sliding is being initiated. Also, both rectangles change in shape and distort following the measured lateral displacements. Figure 24a shows what happen at 5.5 seconds, which is close to the end of the weak shaking, when there was already some pore pressure buildup in the soil but very small shear strains. Figure 24b at t = 7.0 seconds shows the end of weak shaking, where there is some pore pressure buildup in the soil (about 30% in the middle of the deposit), but essentially no strains yet. Figure 24c at t = 8.25 seconds, shows the first zone of concentrated strain at a shallow depth of about 2 m, indicating that the first block has started to slide. Figure 24d at t = 8.5 seconds, shows the second zone of concentrated strain near the base of the soil, indicating that the second block has started to slide.

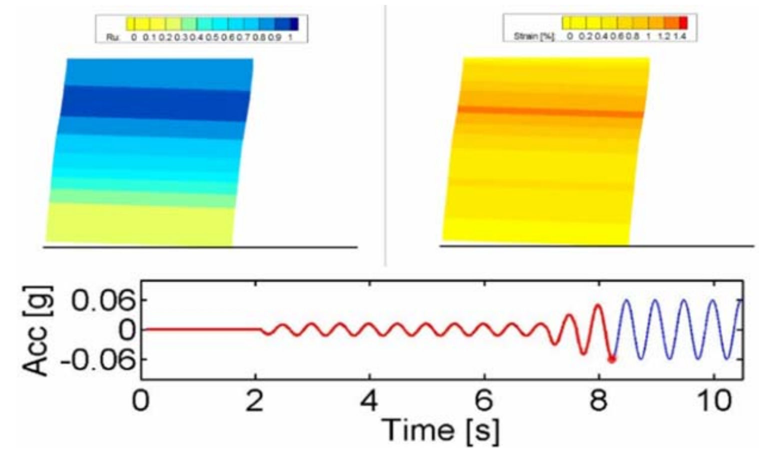
Conclusions

- We are starting to understand better the detailed mechanics of lateral spreading and its interaction with pile foundations.
- Some significant parameters are:
    - Free field permanent ground deformation .
    - Existence and features of the shallow nonliquefiable layer.
    - Lateral stiffness (and strength) of pile foundation system including superstructural constraints.
    - Areas of pile foundation exposed to soil lateral pressures .
    - Permeability of liquefiable layer (p-y curves).

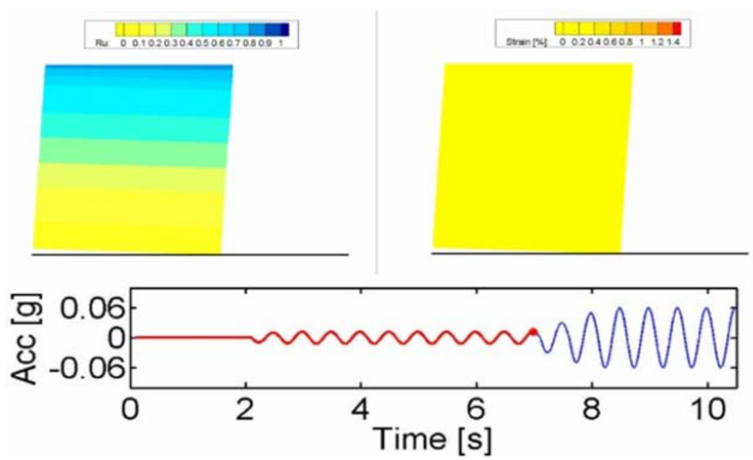
- There is currently active research in several countries, combining field observations and testing, 1g and centrifuge tests, numerical simulations, and use of advanced sensors.



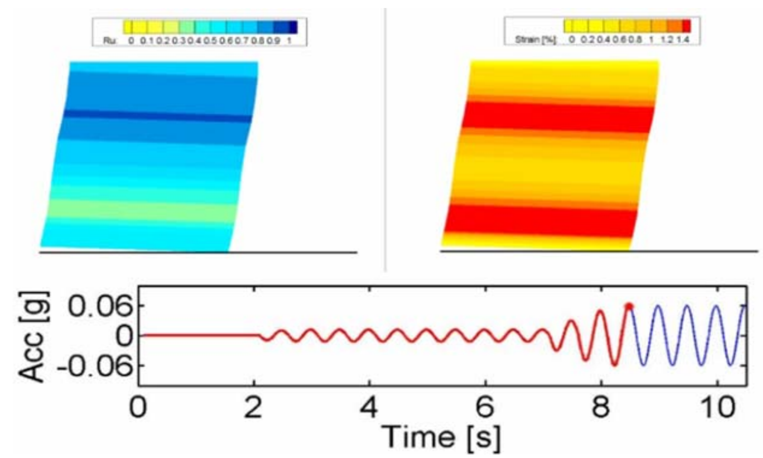
(a) t = 5.5 sec., close to the end of weak shaking, some pwp, no permanent shear straining.



(c) t = 8.25 sec., zone (red) of first concentrated straining at a shallow depth of about 2 m showing that first block slide has started.



(b) t = 7.0 sec., end of weak shaking, some pwp, essentially no permanent shear straining.



(d) t = 8.5 sec zone (red) of second concentrated straining near the base of the soil showing that second block slide has started.

Figure 24. Snapshots of visualization of Test SG1 at four different times during shaking (Ubilla, 2007).

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