4th International Conference on Earthquake Engineering
Taipei, Taiwan
October 12-13, 2006

Paper No. 42

LIQUEFACTION-INDUCED LATERAL LOAD ON PILES

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ABSTRACT

A shake-table series of experiments has provided valuable data for liquefaction-induced lateral spreading effects on pile foundations. In this paper, this data is employed to estimate peak soil pressure on single piles embedded in a laterally spreading liquefied layer, and to calibrate a nonlinear elasto-plastic computational model, within the open framework for simulation OpenSees. On this basis, a user interface is under development, to facilitate numerical studies by interested researchers worldwide.

Keywords: Piles, Liquefaction, Lateral spreading, Earthquakes, Soil-structure interaction, Shake table, Numerical modeling.

INTRODUCTION

Seismic response of pile foundations in liquefying soil is currently the subject of major research in geotechnical earthquake engineering (Dobry and Abdoun, 2001; Finn and Fujita, 2002; Boulanger and Tokimatsu, 2005; Liyanapathirana and Poulos, 2005a, b). Observed pile damage and failure due to seismic excitation (e.g., Hamada and O'Rourke, 1992; Tokimatsu and Aska, 1998) are necessitating an increased understanding through experimental study including centrifuge tests (e.g., Abdoun, 1997; Haigh, 2002; Abdoun et al., 2003; Bhattacharya, 2003; Imamura et al., 2004; Brandenberg et al., 2005), one-g shake table experiments (e.g., Tokia et al., 1993; Hamada, 2000; Meneses et al., 2002; Tokimatsu and Suzuki, 2004; Cubrinovski et al., 2006; Dungca et al., 2006; Towhata, 2006), and full-scale field tests using controlled blast (e.g., Ashford et al., 2006). Analytical expressions and analysis and design procedures for estimating liquefaction-induced lateral load on piles are being currently developed on this basis (e.g., O'Rourke et al., 1994; ATC and MCEER, 2001; JRA, 2002; Dobry et al., 2003; Cubrinovski and Ishihara, 2004; Rollins et al., 2005; Weaver et al., 2005; Juirnarongrit and Ashford, 2006).

Many of the conducted experimental studies have been focused on lateral spreading loads due to a liquefying layer with or without an upper non-liquefiable stratum (e.g., Wilson et al., 2000; Abdoun et al., 2003; Brandenberg et al., 2005; Ashford et al., 2006). Indeed, much damage has been attributed to

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these two important scenarios as suggested by case history investigations (e.g., Hamada, 1992; Hamada and O'Rourke, 1992; Tokimatsu and Aska, 1998; Berrill et al., 2001).

The above studies have provided valuable field and experimental data to calibrate analysis and design procedures, and have brought insight into the complex mechanisms of pile response during lateral spreading. To further investigate pile behavior in the scenario of a mildly sloping liquefiable ground, a new series of shake-table experiments was conducted recently to provide additional data (He, 2005; He et al., 2006). Single pile and small pile groups are subjected to liquefaction-induced lateral spreading with and without an upper non-liquefiable stratum. Liquefied layers ranged from 1.7 m to 5.5 m in thickness in this series of experiments.

The data of this new series of experiments is now available for study (http://it.nees.org). Herein, attention will be focused on the estimation of peak soil pressure on single piles embedded in a laterally spreading liquefied layer. Calibration of a nonlinear elasto-plastic computational model is also undertaken, within the open framework for simulation (OpenSees). On this basis, a user interface is under development, to allow further numerical studies by interested researchers worldwide. The components of this interface are briefly described in this paper.

DESCRIPTION OF THE ONE-G SHAKE TABLE EXPERIMENTS

A total of seven shake table experiments were conducted. Among these experiments, four (Models 1-4) were conducted using a large size laminar box at the National Research Institute for Earth Science and Disaster Prevention (NIED) laboratory in Tsukuba, Japan, and three (Models 5-7) were conducted using a medium size laminar box at the University of California, San Diego (UCSD).

The sand stratum in the models was constructed by the sedimentation method (sand deposition in water). Relative density was about 40% - 50% and saturated density was about 1940 kg/m^3. Each model was instrumented with accelerometers and pore pressure sensors within the soil. Displacement transducers were mounted on the laminar box exterior wall to measure free-field lateral displacements. The piles were instrumented with strain gages and displacements transducers to measure bending moment and deformation during shaking.

The piles in all experiments were fixed to the base in an attempt to generate a fixed cantilever boundary condition. Each pile was densely instrumented with strain-gages to measure induced bending moments and deformation in the pile during lateral spreading. Static pushover tests were conducted before soil layer construction to obtain the bending stiffness EI and the actual base fixity condition of the piles. Table 1 summarizes properties of the soil layers and pile foundations. The experiments are briefly described below.

Experiments Using the Large Size Laminar Box

The four experiments (Models 1-4) conducted at NIED employed a large laminar box (Figure 1) which was inclined at 2º to the horizontal, patterned after Abdoun et al. (2003) and Dobry et al. (2003). The employed laminar box is about 12 m long, 6 m high and 3.5 m wide (Kagawa et al., 2004). Figure 2 shows the test setup of Model 4 (He, 2005). Model 1 consisted of a 5.5 m sand layer with a water table at the downslope ground surface. Mode 4 consisted of a 5.0 m sand layer with a water table at the upslope ground surface. The setup of Modes 2 and 3 were the same as Models 1 and 4, respectively, with the water table in Mode 2 and Model 3 one meter below the downslope ground surface.

In Models 1 and 2, a single pile and a 2x2 pile group were tested. The single pile made of steel pipe 31.8 cm in diameter and 6 mm in wall thickness was at the upslope about 3.4 m from the downslope pile group. In Models 3 and 4, two separate single piles with different stiffnesses were tested. The relatively stiff pile had the same properties as the single piles in Models 1 and 2. The relatively flexible pile was also made of steel pipe and had a diameter of 31.8 cm and a wall thickness of 3 mm.

Table 1. Summary of the soil profiles and pile foundations during the shake table experiments.

Test	Soil profile		Pile Properties				
	Height (m)	Water Table	Embedded Length (m)	Diameter (cm)	Wall Thickness (mm)	Bending Stiffness EI (kN·m^2)	Base Fixity* (kN·m/rad)
1	5.5	At downslope ground surface	5.5	31.8	6	14320	18500
2	5.5	1.0 m below ground surface	5.5	31.8	6	14320	18500
3	5	1.0 m below ground surface	5.0	31.8	6	14320	18500
			5.0	31.8	3	7360	8500
4	5	Covers the entire soil layer	5.0	31.8	6	14320	18500
			5.0	31.8	3	7360	8500
5	1.89	Covers the entire soil layer	1.70	25.0	6.4	120	110
6	1.75		1.56	25.0	6.4	2600	200
7	1.71		1.52	25.0	6.4	2600	200

*Pile base fixity condition is characterized by a rotational spring with constant stiffness

Figure 1. The NIED large size laminar box (Kagawa et al., 2004).

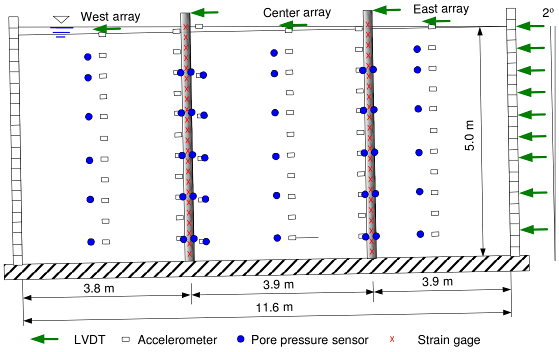


Figure 2. Test setup of Model 4 (He et al., 2006).

Experiments Using the Medium Size Laminar Container

The three experiments (Models 5-7) conducted in a medium size laminar box at UCSD included a single pile each. The employed soil box (Figure 3) about 4 m long, 2 m high and 1.8 m wide (Jakrapiyanun, 2002) was also inclined at 2° to the horizontal. Figure 4 shows the test setup and instrumentation of Model 7. The other tests, Models 5 and 6 had a similar setup and instrumentation pattern.

Model 5 included a plastic pile which was relatively flexible compared to the aluminum pile in Models 6 and 7. All piles had a diameter of 25 cm and a wall thickness of 6.4 mm. The piles were installed in the center of the laminar box.

Dynamic Excitation

Shaking of the models was carried out along the sloping direction. Input motions of the experiments were sinusoidal accelerations with different frequencies and amplitudes (Table 2). In particular, input motions in Models 1, 3, and 4 were mainly at a frequency of 2 Hz and 0.2 g amplitude, with Model 2 excitation consisting of a 2 Hz and 0.3 g sinusoidal wave. Models 5-7 were shaken with a 1 Hz, 0.15 g sinusoidal acceleration. Shaking duration varied from 14 to 70 seconds as shown in Table 2.

Figure 3. The UCSD medium size laminar box (Jakrapiyanun, 2002).

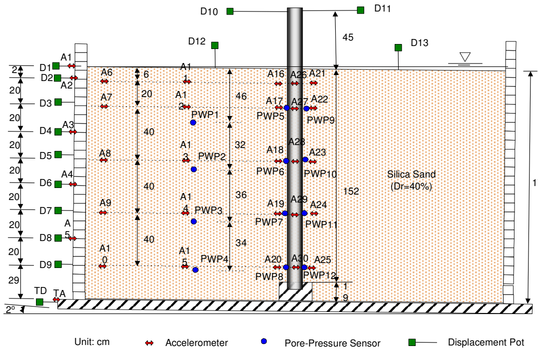


Figure 4. Test setup of Model 7 (He, 2005).

Table 2. Summary of employed input motions.

Model Number	Input Motion		
	Frequency (Hz)	Amplitude (g)	Duration (s)
1	2	0.2	14
2	2	0.3	14
3	2	0.2	44
4	2	0.2	44
5	1	0.25	15
6	1	0.25	35
7	1	0.25	70

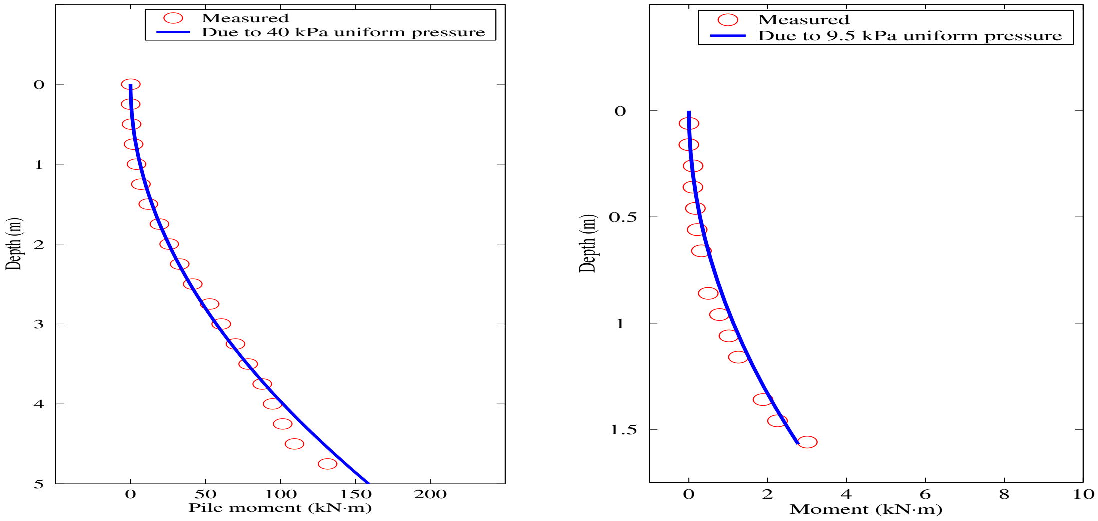
TEST RESULTS

Soil response (acceleration, displacement, and excess pore pressure) and pile response (pile head displacement and strains along the pile) were measured during shaking. Detailed testing results are discussed in He (2005). This paper presents results of a preliminary analysis of lateral pressure on the piles at the instant of peak pile moment.

For this purpose, pile bending moment was first calculated based on the measured strain using the traditional Euler-Bernoulli beam theory. The critical time step when maximum moment occurred in the piles was identified from the moment time histories. Subsequent analyses focusing on lateral pressure employed pile moments and displacements at this critical time step.

In this preliminary analysis, a uniform lateral pressure (Dobry et al., 2003) was back-calculated for each pile allowing the best match of measured peak moment profile. Figure 5 shows maximum pile moment profiles measured during Models 4 and 6, and those estimated using the back-calculated pressure. The back-calculated uniform pressure for all piles is shown in Table 3. It can be seen from Table 3 that a different level of pressure is required for each case. Lateral pressure on piles within the thick liquefied layer is significantly larger than that within the shallow stratum.

Of interest is that the uniform soil pressure from the large laminar box experiments (thick liquefied soil layer) is in the range from 20 to 40 kPa, higher than some earlier recommendations (e.g., Dobry et al., 2003) derived from centrifuge experiments with water as the pore-fluid. Recent centrifuge experiments conducted with a viscous pore fluid also showed a large equivalent lateral uniform pressure on the pile in a range from 16 kPa (Haigh, 2002; Haigh and Madabhushi, 2002) to 33 kPa (Gonzalez et al., 2005).



(a) Stiff pile in Model 4 (b) Aluminum pile in Model 6

Figure 5. Measured and estimated pile moments.

Table 3. Summary of the pile and ground responses at the instant of peak moment.

Model Number	**Maximum pile response**			**Free-field ground surface displacement**		**Soil pressure**[**] (kPa)
	Pile	M^*_{max} (kN·m)	Pile head deflection (cm)	At the same time as M_{max} (cm)	At end of shaking (cm)	
1	Stiff pile	86	10	7	27	22
2	Stiff pile	118	12	8	15	25
3	Stiff pile	166	14	19	52	40
	Flexible pile	95	25	43		40
4	Stiff pile	132	11	13	105	40
	Flexible pile	95	21	15		40
5	Single pile	2.65	4.2	8.5	12.5	7
6	Single pile	3.00	1.6	2.8	7.8	9.5
7	Single pile	2.83	1.2	1.9	2.5	9.0

[*] Near base except for flexible piles (at 4 m depth) in view of yielding near the pile base.
[**] Uniform soil pressure along the pile based on the M_{max} moment profile.

NUMERICAL SIMULATION

OpenSees

The above data along with earlier centrifuge testing data sets is being employed to calibrate a nonlinear elasto-plastic computational model, within the Pacific Earthquake Engineering Research (PEER) Center OpenSees Framework (developed under the leadership of Professor Gregory Fenves of UC Berkeley). OpenSees is a software framework for developing applications to simulate the performance of structural and geotechnical systems subjected to earthquakes (Mazzoni et al., 2006).

The Constitutive Model

The soil constitutive model (Parra, 1996; Yang and Elgamal, 2002; Elgamal et al., 2003) implemented in OpenSees was developed based on the original multi-surface-plasticity theory for frictional cohesionless soils (Prevost, 1985). This model (Figures 6 and 7) was developed with emphasis on simulating the liquefaction-induced shear strain accumulation mechanism in clean medium-dense sands (Yang and Elgamal, 2002; Elgamal et al., 2003). Special attention was given to the deviatoric-volumetric strain coupling (dilatancy) under cyclic loading, which causes increased shear stiffness and strength at large cyclic shear strain excursions (i.e., cyclic mobility).

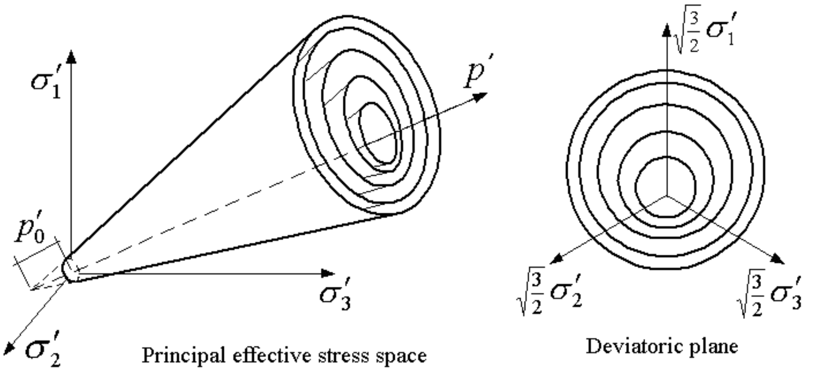


Figure 6. Conical yield surfaces for granular soils in principal stress space and deviatoric plane (after Prevost, 1985; Yang et al., 2003).

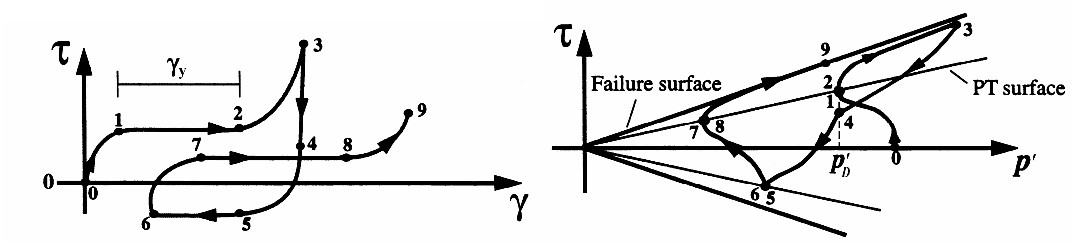
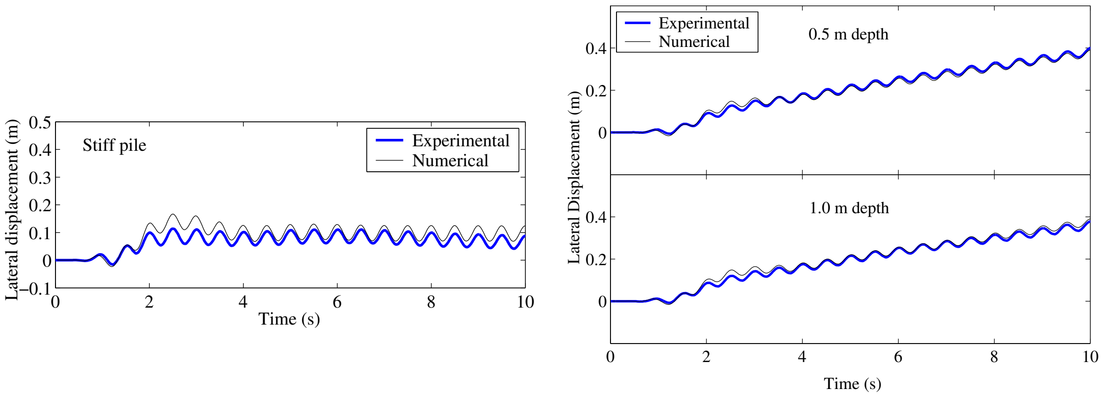


Figure 7. Shear stress-strain and effective stress path under undrained shear loading conditions (Yang et al., 2003).

Model Calibration

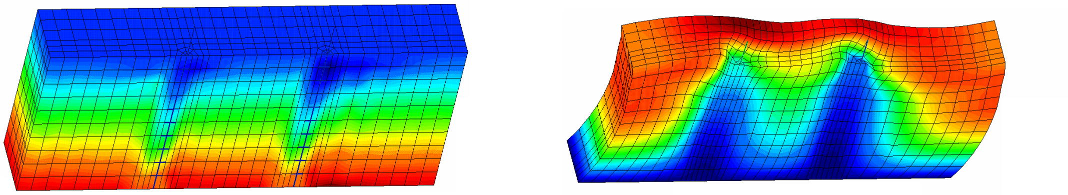
Results of Model 4 were employed to assess possible ranges and significance of the model parameters, through finite element simulations. The main modelling parameters include typical dynamic soil properties such as low-strain shear modulus, friction angle, and permeability, as well as calibration constants to control pore-pressure buildup rate, dilation tendency, and the level of liquefaction-induced cyclic shear strain. Pile properties including bending stiffness and base fixity were obtained from the static pushover tests as discussed earlier. The computed response of Model 4 (stiff pile) along with the experimental response is shown in Figure 8.

In general terms, this series of experimental data along with finite element simulation currently suggest an apparent pinning effect of the two piles in the container (Model 4), little dilative tendency (during this shaking event), and lower overall excess pore pressure near the pile compared to the free-field.



Recorded and computed pile head displacement time histories

Recorded and computed free-field displacement time histories



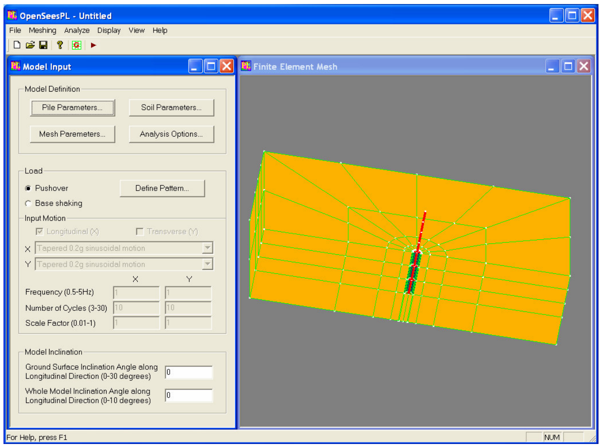
Sketch of excess pore pressure at 10 seconds

Sketch of deformed mesh at 10 seconds

Figure 8. Computed and Experimental Response of Model 4 (He, 2005)

USER INTERFACE

A user interface "OpenSeesPL" is under development (Figure 9), to allow for the execution of single pile simulations under seismic excitation scenarios as well as for pushover studies (Lu et al., 2006). The Finite Element analysis engine for this interface is the OpenSees Framework (Mazzoni et al., 2006).



● Pushover
○ Base shaking
☑ Longitudinal (X)
☐ Transverse (Y)
X Tapered 0.2g sinusoidal motion
Y Tapered 0.2g sinusoidal motion

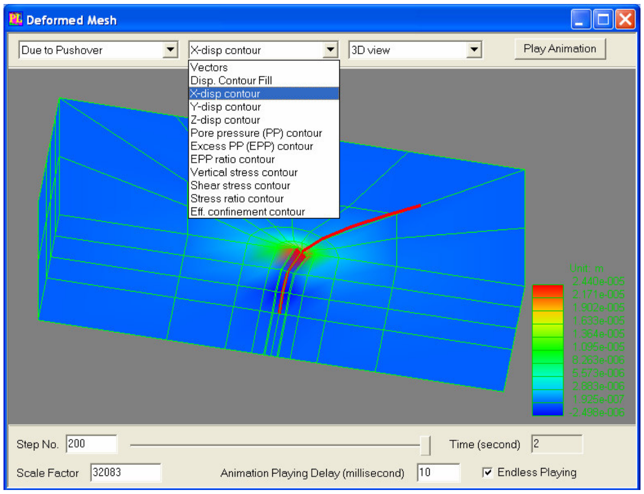
	X	Y
Frequency (0.5-5Hz)	1	1
Number of Cycles (3-30)	10	10
Scale Factor (0.01-1)	1	1

Model Inclination
Ground Surface Inclination Angle along Longitudinal Direction (0-30 degrees): 0
Whole Model Inclination Angle along Longitudinal Direction (0-10 degrees): 0

Figure 9. OpenSeesPL user interface with mesh showing a circular pile in level ground (view of ½ mesh employed due to symmetry for uni-directional lateral loading).

OpenSeesPL includes a pre-processor for: 1) definition of the pile geometry (circular or square pile) and material properties (linear or nonlinear), 2) definition of the 3D spatial soil domain (with uniform soil properties for each layer laterally), 3) definition of the boundary conditions and input excitation or push-over analysis parameters, and 4) selection of soil materials from an available menu of cohesionless and cohesive soil materials (Table 4). The menu of materials (Table 4) includes a complementary set of modeling parameters representing loose, medium and dense cohesionless soils (with silt, sand or gravel permeability), and soft, medium and stiff clay (J_2 plasticity cyclic response model). Representative soil properties are pre-defined for each of these soils (Table 4).

OpenSeesPL allows convenient pre-processing and graphical visualization of the analysis results including the deformed mesh (Figure 10), ground response time histories and pile responses. This interface is designed for simplicity, and is intended to be intuitive and self-explanatory. OpenSeesPL makes it possible for geotechnical and structural engineers/researchers to build a model, run the finite element analysis and evaluate performance of the pile-ground system (Lu et al., 2006).



Step No. 200

Time (second) 2

Scale Factor 32083

Animation Playing Delay (millisecond) 10

☑ Endless Playing

Figure 10. Graph types available in the deformed mesh window.

Table 4. Representative set of basic material parameters (data based on Seed and Idriss (1970), Holtz and Kovacs (1981), Das (1983), and Das (1995)).

Cohesionless Soils	Shear wave velocity* at 10m depth (m/s)	Friction angle (degrees)	Possion's ratio	Mass density (kg/m^3)
Loose	185	29	0.4	1.7x10^3
Medium	205	31.5	0.4	1.9x10^3
Medium-dense	225	35	0.4	2.0x10^3
Dense	255	40	0.4	2.1x10^3
Cohesive Soils	Shear wave velocity (m/s)	Undrained shear strength (kPa)	Possion's ratio	Mass density (kg/m^3)
Soft clay	100	18.0	0.4	1.3x10^3
Medium clay	200	37.0	0.4	1.5x10^3
Stiff clay	300	75.0	0.4	1.8x10^3

* Shear wave velocity of cohesionless soils in proportion to $(p_m)^{1/4}$ where p_m is effective mean confinement.

CONCLUSIONS

A series of one-g shake-table experiments on piles subjected to lateral spreading was conducted using a mildly inclined laminar box. A uniform soil pressure based on the measured peak moment profile was back-calculated. It was found that a different level of pressure was required for each case. Lateral pressure on piles within a thick liquefied layer was significantly larger than for cases of shallow strata. The uniform soil pressure in the large laminar box experiments (liquefied soil layer up to 5.0-5.5 m) was in the range of 20 to 40 kPa, a value considerably higher than some current recommendations. Further analyses are required to better characterize lateral load on piles due to liquefaction-induced lateral spreading.

Upon calibration, the finite element analysis produced a reasonable match of pile and soil responses. This calibration process suggested the two piles (Model 4) had apparent pinning effects on the soil stratum. These effects significantly reduced the ground displacement. Excess pore pressure was somewhat lower near the pile than in the free-field. Under the imparted excitation, the model exhibited little dilative tendency.

In an attempt to increase efficiency and reduce the chance for error, a user-friendly interface is being developed to facilitate use of otherwise complicated computational environments with numerous (often vaguely defined) input parameters. The effort is a first step in the direction of allowing for more convenient exposure and utilization of such computational tools. A peer review process is needed to verify and provide extra credibility to the pre-defined structural and soil model parameters and the resulting response. In a more general framework, the process can facilitate collaborative efforts, and comparisons between constitutive models and numerical formulations of different researchers, as envisioned by the UC Berkeley OpenSees platform developments.

ACKNOWLEDGEMENTS

This research was supported by the Pacific Earthquake Engineering Research Center (PEER), under the Earthquake Engineering Research Centers Program of the National Science Foundation (award No. EEC-9701568), and by the National Science Foundation (Grants No. CMS0084616 and CMS0200510) and the PEER Lifelines Program. The mildly inclined laminar box testing configuration is patterned after the earlier centrifuge research effort of Rensselaer Polytechnic Institute (Professors Ricardo Dobry and Tarek Abdoun). We are grateful to Professor Scott Ashford (UCSD) for providing the laminar container. The experiments using the large laminar box were conducted at the National Research Institute for Earth Science and Disaster Prevention (NIED) laboratory in Tsukuba, Japan (Dr. Akio Abe, Dr. Masayoshi Sato, and Professor Kohji Tokimatsu). Professors Ross Boulanger (UC Davis), Liam Finn (UBC), and Tzou-Shin Ueng (NTU) acted as advisors to this project and provided valuable suggestions.

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