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Behavior of Pile Foundations in Liquefied and Laterally Spreading Ground

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ABSTRACT

Methods for predicting the performance of pile foundations in liquefying and laterally spreading ground during earthquakes have developed considerably in recent years. Nonetheless, the mechanisms of soil-pile interaction in liquefied soil are still not well understood and the accuracy of design methods remains to be quantified. Subsequently, a series of dynamic centrifuge model experiments are being performed at UC Davis to study the behavior of single piles and pile groups in a soil profile comprised of a nonliquefied crust spreading laterally over a loose saturated sand layer. This paper will discuss some recent findings on the lateral resistance of liquefied soil, present initial evaluations of simplified pushover design methods against centrifuge test data, and summarize current recommendations for engineering practice.

INTRODUCTION

Extensive damage to pile-supported structures in areas of liquefaction and lateral spreading has been observed in many earthquakes around the world. A review of case histories and physical modeling studies shows that many important lessons and insights have been learned in recent years, but that numerous questions remain regarding the mechanisms of soil-pile interaction in liquefied soil for many situations. In addition, the accuracy of our evolving design methods remains to be quantified.

Predicting the behavior of a pile foundation in liquefying ground during an earthquake requires consideration of design motions, free-field site response, superstructure response, and soil-pile-superstructure interaction. Evaluating pile performance requires consideration of the inertial and kinematic loads imposed on the piles and their pile-cap connections, transient or permanent deformations of the pile foundation, the influence of the pile foundation on the dynamic

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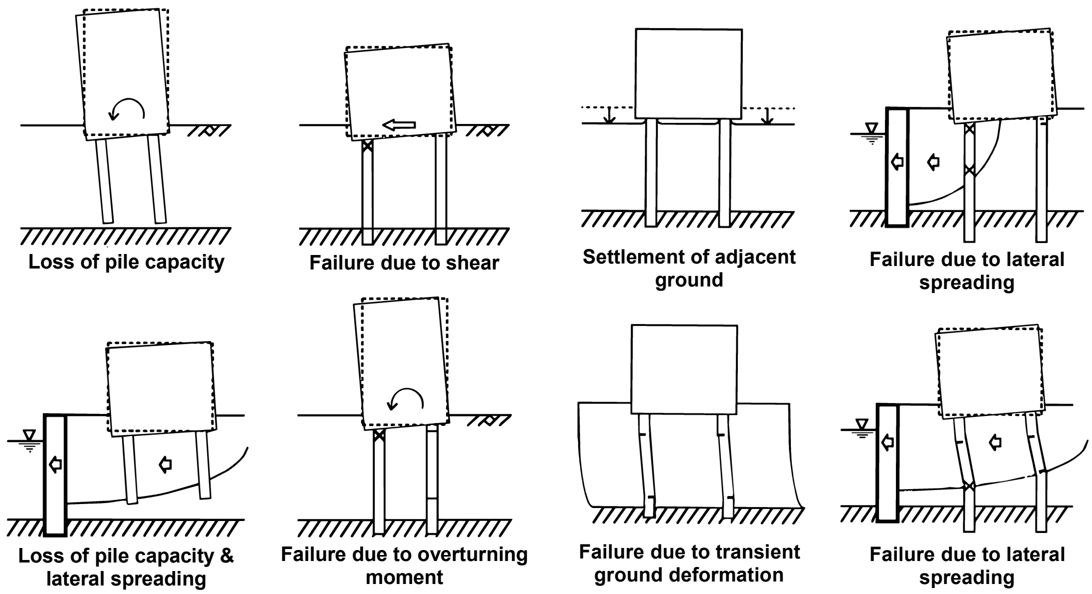


Fig. 1. Schematics of pile damage mechanisms in liquefied ground (modified from Tokimatsu et al. 1996).

response of the superstructure, and the performance criteria for the pile foundation. Quantifying the effects of liquefaction on these and other aspects of the soil-structure interaction problem continues to be a challenging task despite the advances of recent years.

Different mechanisms of damage to pile foundations are illustrated in Figure 1 for cases with and without lateral spreading. Both inertial and kinematic loading must be considered, with the appropriate load combination varying as liquefaction develops during shaking. Kinematic loading will vary with the magnitude of ground deformations and the strength/stiffness of the soil during a given loading cycle. Peak ground deformations can occur either during or toward the end of shaking, depending on the magnitude of transient ground movements (lurching) during the lateral spreading process. Considerable judgment is involved in estimating the appropriate combination of kinematic and inertial loads, and the governing case may be different for the substructure and superstructure.

Simplified design procedures for pile foundations in laterally spreading ground include limit equilibrium methods (e.g., Dobry and Abdoun 2001) and beam on nonlinear Winkler foundation (BNWF) methods. The former approach applies lateral pressures against the pile that are independent of the free-field displacement, which is reasonable when the free-field displacements are large enough for the lateral soil pressures to reach their limiting values. In the latter approach, the free-field site response (e.g., dynamic or permanent deformations) are estimated separately, and then input to the BNWF model. These monotonic “pushover” methods are schematically compared in Figure 2, and are intended to envelop the actual cyclic loading response during earthquake shaking. Applying

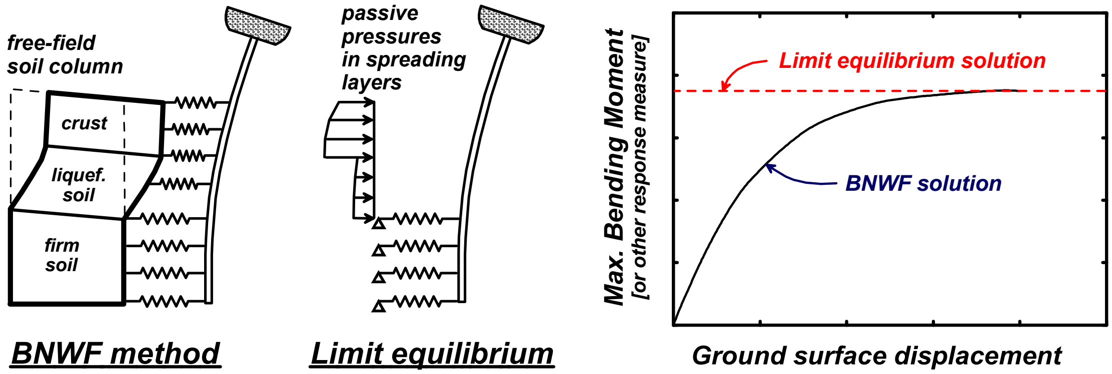


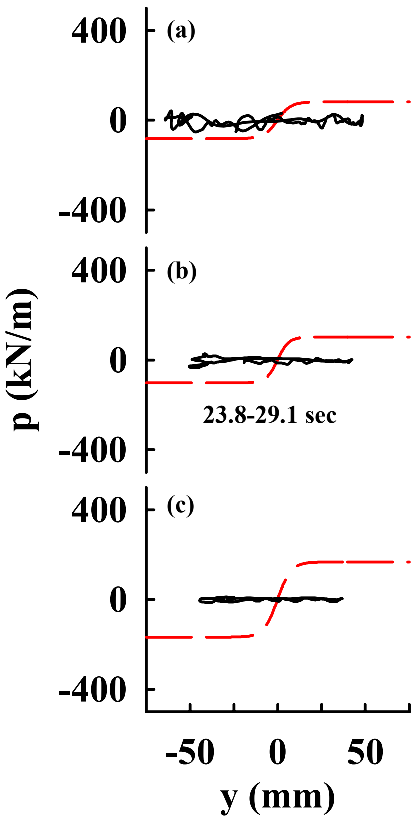
Fig. 2. Schematic comparison of limit equilibrium and BNWF methods.

these approaches to liquefaction problems is complicated by our lack of knowledge of how liquefaction affects the “p-y” behavior of the liquefied soil or an overlying crust (they are usually uncoupled to simplify analyses) and the uncertainty in modeling the free-field response of liquefied deposits. The predictive capabilities and inherent limitations of either analysis method are not yet fully understood, and the resulting uncertainty affects the cost of building new foundations and remediating hazards at existing foundations.

Research at UC Davis on the performance of pile foundations in laterally spreading ground has several ongoing components. A review of case histories and prior physical modeling studies identified several major lessons and insights in the mechanisms of interaction. Subsequently, dynamic centrifuge experiments are being performed to study the behavior of single piles and pile groups in soil profiles comprised of a nonliquefied crust spreading laterally over a loose saturated sand layer. The experimental data are then being used to back-calculate time histories of the lateral pressures against the piles, thereby gaining insight into how the load transfer evolves as liquefaction develops during shaking, and to evaluate the abilities of simplified pushover design methods and nonlinear dynamic FEM analyses to approximate the centrifuge model results. This paper will, however, be limited to a discussion of some recent findings on the lateral resistance of liquefied soil, present initial evaluations of simplified pushover design methods against centrifuge test data, and summarize current recommendations for engineering practice.

LATERAL RESISTANCE OF LIQUEFIED SOIL

The p-y response of liquefying sand is only crudely approximated in simplified design methods that use monotonic envelopes to approximate the truly cyclic behavior. In one of the earliest centrifuge studies of this problem, Dobry et al. (1995) showed that pile bending moments could be reasonably predicted if the original nonliquefied p-y curves were multiplied by an apparent p-multiplier that



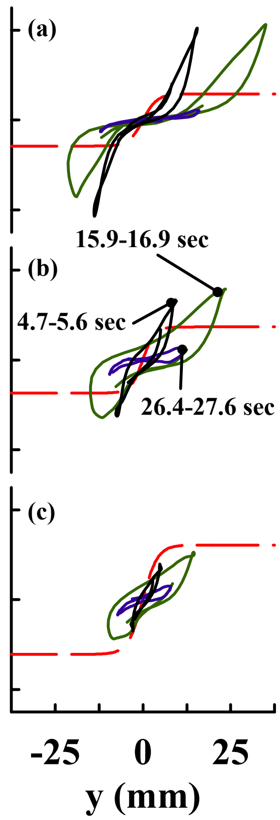


Fig. 3. p-y loops in liquefying loose sand ($D_r \approx 40\%$) at depths (a) 2-D, (b) 3-D, and (c) 4-D (D = 0.67 m). Dashed lines per API (1993). (Wilson et al. 2000)

Fig. 4. p-y loops for liquefying med. sand ($D_r \approx 55\%$) at depths (a) 2-D, (b) 3-D, and (c) 4-D (D = 0.67 m). Dashed lines per API (1993). (Wilson et al. 2000)

decreased more or less linearly with excess pore pressure ratio and reached a minimum value of about 0.1 when the excess pore pressure ratio was unity. Wilson et al. (1999) analyzed the dynamic response of piles in centrifuge tests and concluded that a reasonable p-multiplier for representative peak loading cycles on a single pile in liquefied sand may be about 0.1-0.2 for $D_r \approx 35\%$ and about 0.25-0.35 for $D_r \approx 55\%$. They also showed that the apparent p-y resistance was strongly affected by excess pore pressure variations and soil-pile loading history during shaking, and that peak bending moments and/or peak superstructure displacements may occur before or after liquefaction develops, and thus both conditions need to be considered.

The first measurements of dynamic p-y behavior for liquefying sand were presented by Wilson et al. (2000) based on back-analyses of dynamic centrifuge model tests. Results showed that 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 in Figures 3 and 4. The p-y resistance of loose sand (e.g., $D_r \approx 40\%$) was much smaller and softer than for medium-dense sand (e.g.,

$D_r \approx 55\%$). 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 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.

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 affect the monotonic and cyclic loading behavior of saturated sands, just as should be expected. It is also important to note that the lateral resistance against a pile, after the free-field soil has developed an excess pore pressure ratio (r_u) of 100%, is associated with temporary reductions in r_u as the soil goes through phase transformation (the transition to dilatant behavior). Temporary reductions in r_u to values less than 100% occur in the free-field as a consequence of the earthquake shaking and can also occur locally around the pile due to the extra strains imposed on the soil by the pile's relative movement. Thus, the p-y behavior depends on the following, and likely other, factors.

- Relative density (D_r).
- Prior displacement (strain) history.
- Magnitude of cyclic stresses & number of loading cycles imposed on the free-field soil.
- Number of loading cycles between the pile & soil.
- Excess pore pressure ratio.
- Partial drainage and hence loading rate.
- Soil characteristics.
- Pile installation method.
- Pile characteristics.

Given the complexity of the behavior, it is important to recognize that any simplified monotonic p-y relation for liquefied soil is only a crude approximation for a complex time-varying cyclic loading response.

DYNAMIC CENTRIFUGE MODEL TESTS

Centrifuge tests were performed on the 9-m radius centrifuge at UC Davis at a centrifugal acceleration of 38g. Results are presented in prototype units unless otherwise noted. The centrifuge models were comprised of a soil profile that gently sloped toward a channel at one end. The soil profiles had a nonliquefiable crust of clay (C_u = 23 kPa) overlying a layer of loose saturated sand ($D_r \approx 35\%$),

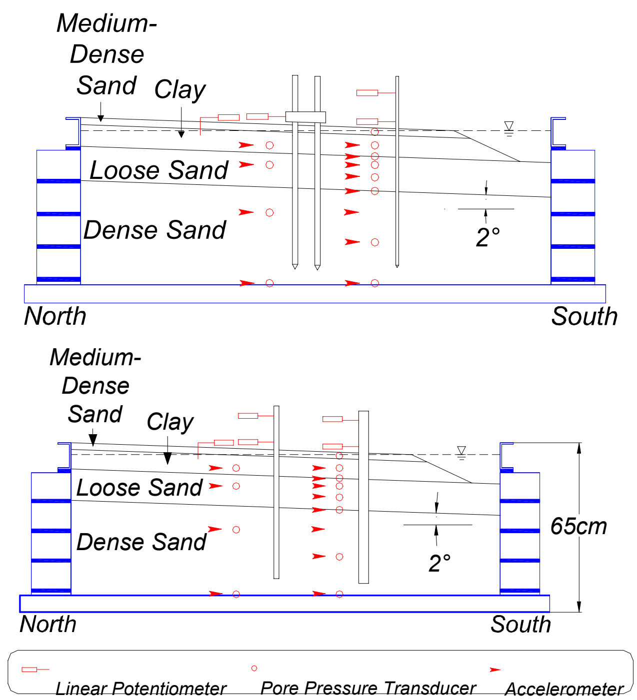


Fig. 5. Cross-sections of centrifuge model (west on top, east on bottom).

overlying dense sand (D_r≈85%). The first centrifuge model had 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 a cap connection for fixed head conditions), located at four separate locations in the model slope. Cross-sections of this model are shown in Figure 5. Subsequent centrifuge models have had a group of six pipe piles connected by a large embedded pile cap. 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.

Only results from the first centrifuge experiment PDS01 (Singh et al. 2000) are described herein. The model was subjected to three earthquakes, separated by sufficient time to allow full reconsolidation of the soil profile. The time histories from all three earthquakes are shown sequentially in the following figures, with the understanding that the time for reconsolidation is not shown. Time histories of acceleration and pore water pressure are presented in Figures 6 and 7. These instruments correspond to two vertical arrays in the soil profile, away from the piles. The excess pore pressure ratio ($r_u = \Delta u/\sigma_{vc}'$), where σ_{vc}' is the vertical consolidation stress, rises to 100% in the upper portion of the loose sand layer (the instrument at 4.1 m depth is near the top of the loose sand layer) and are much smaller in the dense sand layer (as expected). Excess pore pressures also developed in the soft clay layer during the strong shaking, and these records were monitored to ensure full re-consolidation prior to the next shaking event.

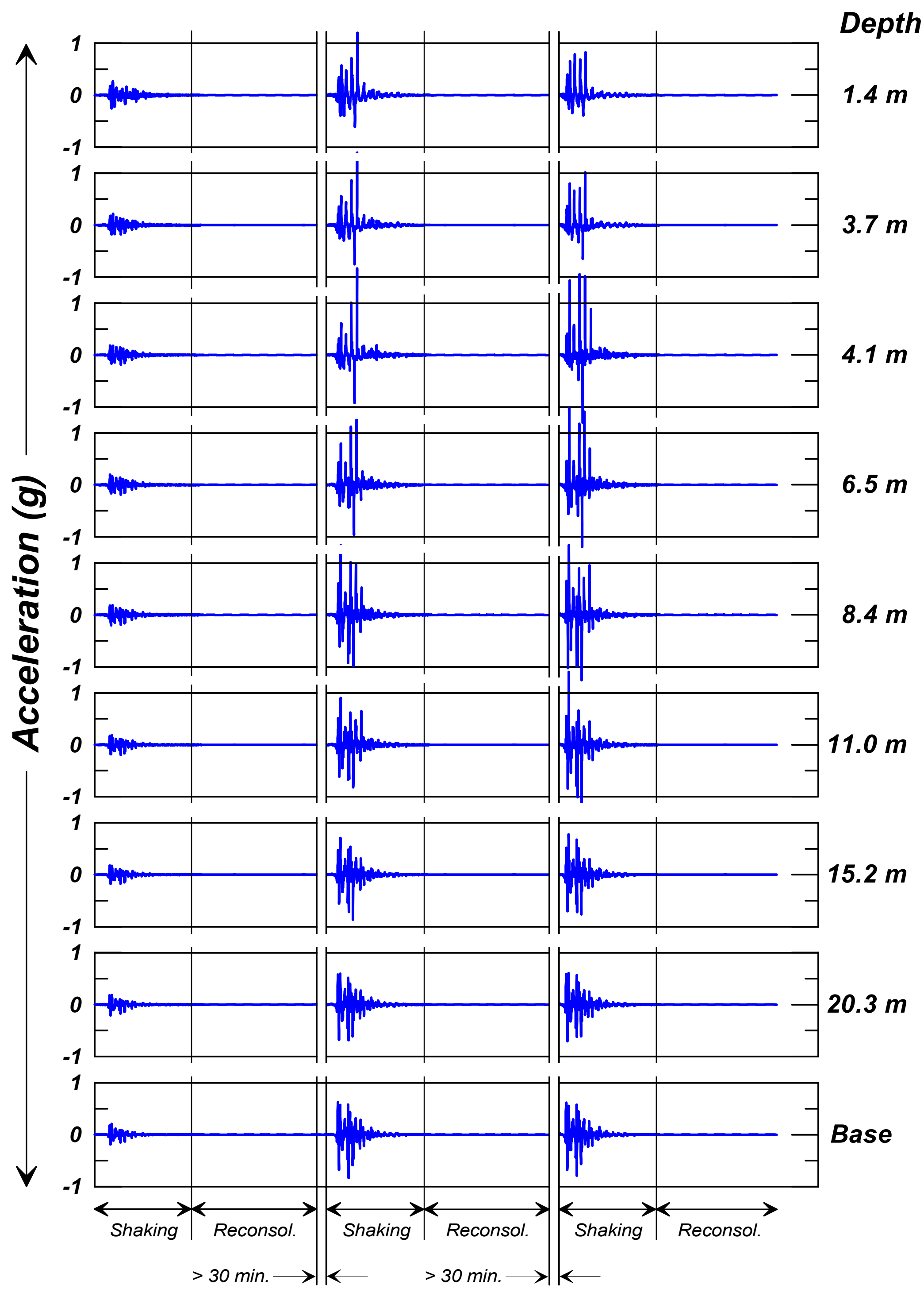


Fig. 6: Acceleration time histories for a vertical array of accelerometers located near the center of the model container for PDS01.

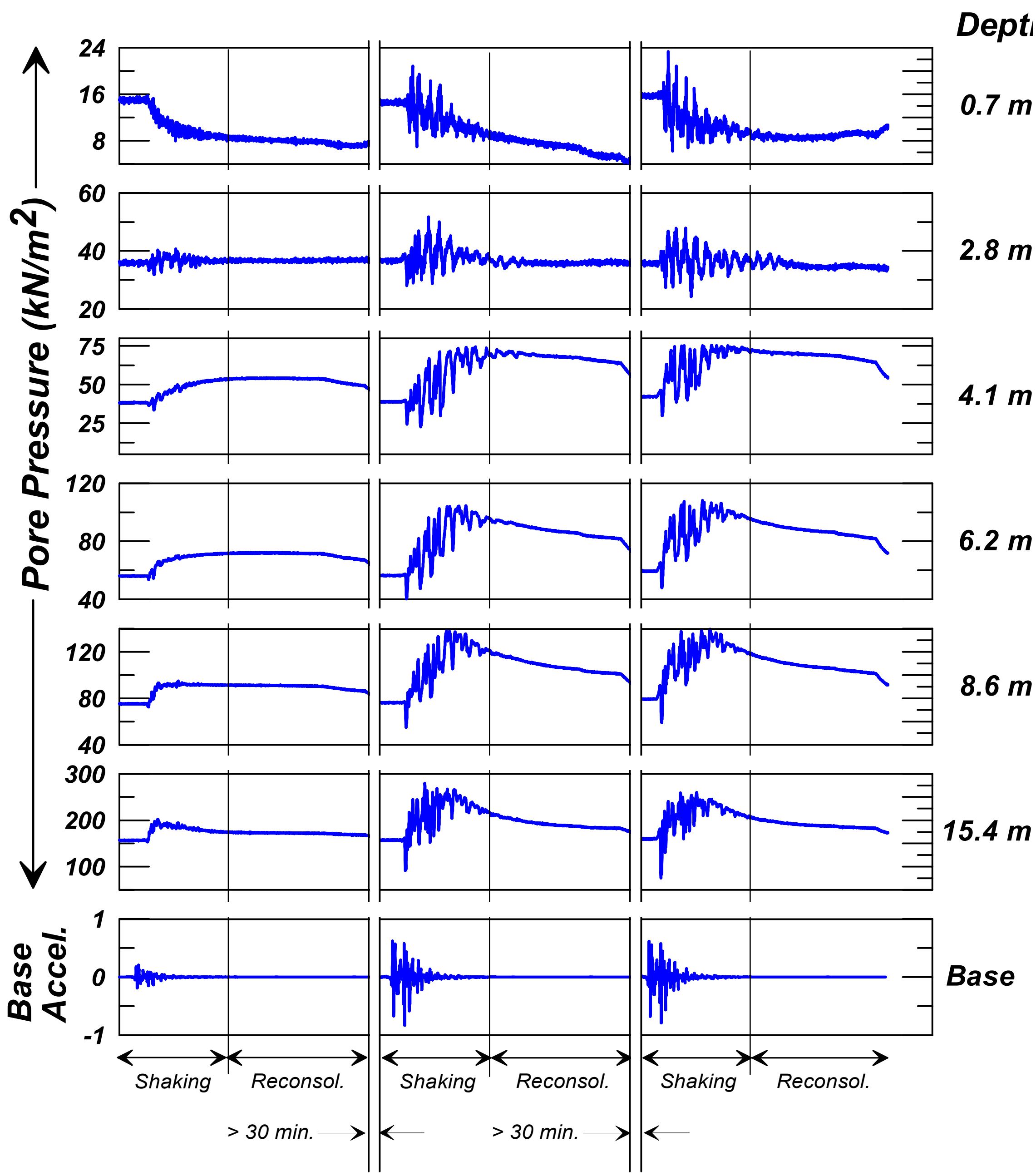


Fig. 7: Pore pressure time histories for a vertical array of pore water pressure transducers near the center of the model container for PDS01.

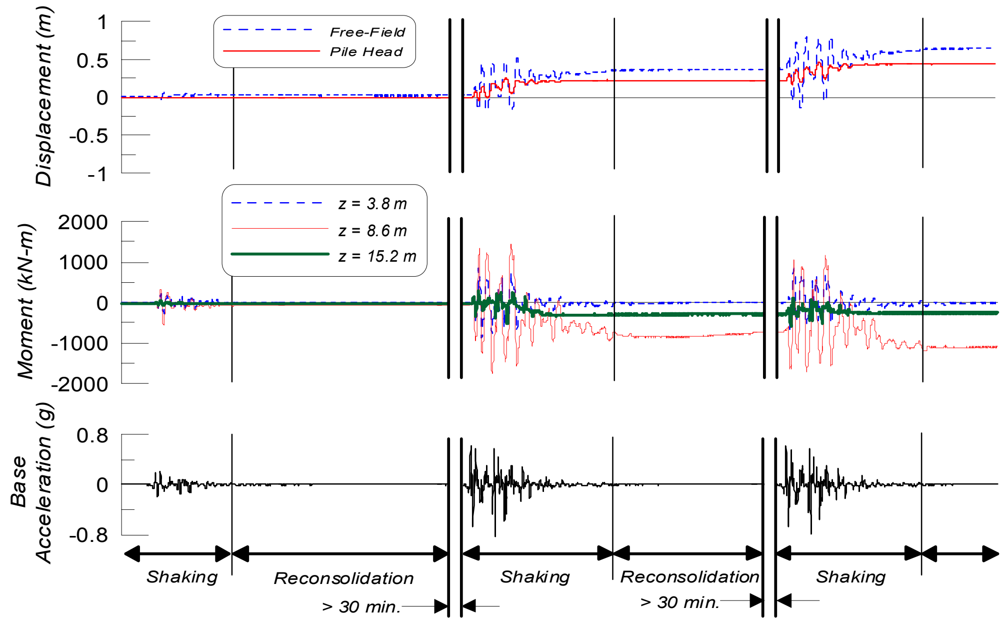


Fig. 8. Displacement, moment and base acceleration time histories for the medium-sized single pile. Z is measured from the ground surface.

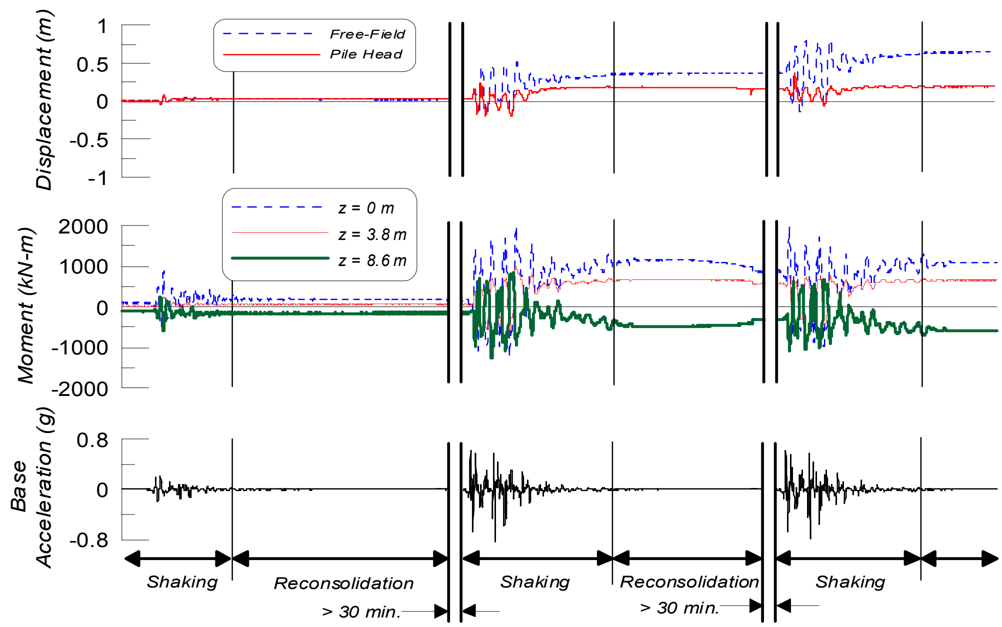


Fig. 9. Displacement, moment and base acceleration time histories for the upslope pile in the 2-pile group. Z is measured from the ground surface.

Time histories of pile head displacement, ground displacement, and pile bending moments are shown in Figure 8 for the 0.73-m diameter pile and in Figure 9 for the group of two 0.73-m diameter piles from the first centrifuge model test. These figures show that large residual displacements and pile bending moments remained after the second earthquake. The clay layer's transient lateral displacements were greater than its residual lateral displacements, and similarly

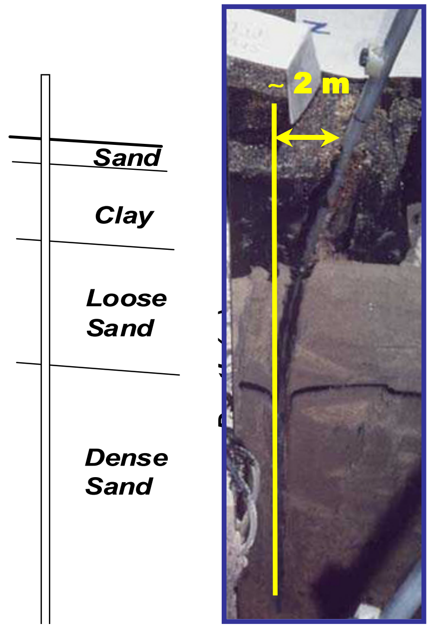


Fig. 10. Photo of deformed 0.36-m-diameter SP pile exposed during excavation of model after testing, with accompanying schematic of soil profile.

the pile's transient bending moments were greater than its residual bending moments. The peak bending moments for these piles were not significantly different in the third earthquake than in the second earthquake, despite the further increase in lateral spreading displacement of the nonliquefied crust and an increase in the peak pile displacements. This result indicates that the full passive resistance of the crust had already been mobilized against the piles during the second earthquake.

The 0.36-m-diameter SP pile yielded during the tests, forming a plastic hinge near the lower portion of the loose sand layer. A photo of the yielded pile, as exposed during the post-test excavation of the model, is shown in Figure 10. The permanent deformation of the pile head was about 2 m in prototype dimensions. The other piles remained elastic during the tests.

PSEUDO-STATIC ANALYSIS OF KINEMATIC LOADING

The experimental results were compared against a pseudo-static pushover-type analysis using a beam on nonlinear Winkler foundation (BNWF) approach, as was schematically illustrated in Figure 2. These analyses were performed using the program LPile$^+$ (Reese et al. 2000). Matlock's (1970) static p-y relation for soft clay and Reese et al.'s (1974) static p-y relation for sand were used. Parameter studies were then used to evaluate the sensitivity of the analysis results to the soil displacement profile, the soil properties, and the p-y relations (Singh 2002).

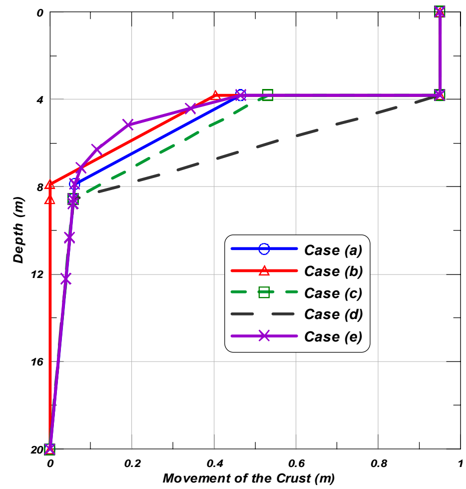


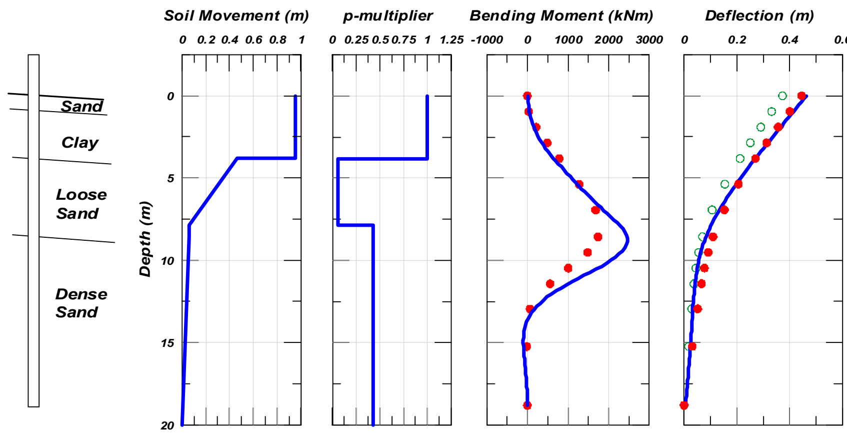
Fig. 11. Different assumed soil displacement profiles for parametric study of pushover analysis method.

The soil displacement profile was represented by five different approximations, as illustrated in Figure 11. Case (e) is the approximation that most closely represents the measured soil displacement profile. Cases (a-d) were different approximations that a designer might reasonably have chosen for the same level of ground surface displacement. It should be noted, however, that very different displacement profiles (i.e., cases with relatively constant shear strains in the liquefied layers, and cases with larger shear strains at the bottom rather than at the top of the liquefied layers) have been observed in numerous other model studies. The shape of the ground deformation is affected by numerous factors (e.g., stratigraphy, permeability contrasts, heterogeneity within layers), and thus the shapes shown in Figure 11 are specific to this centrifuge test and are not to be generalized for design purposes.

The effects of excess pore pressure or liquefaction on the p-y resistance of the sand layers was accounted for as follows. The ultimate lateral resistance (p_{ult}) was assumed to vary linearly with the free-field's 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. The appropriate S/σ_{vc}' ratio has been found to depend on several aspects of the loading condition (as discussed previously), and is being further investigated by back-calculating lateral resistances from these tests. For the purposes of these pushover analyses, the adopted values for S/σ_{vc}' were 0.07 for the loose sand layer and 0.10 for the dense sand layer (even though it did not liquefy, this ratio was needed to make some allowance for the effects of r_u values in the dense sand). The sensitivity of the analysis results to these ratios, as well as the other important input parameters, was evaluated later (Singh 2002). Lastly, the p_{ult} values calculated in the above manner can also be expressed in terms of an equivalent p-multiplier (i.e., by dividing p_{ult} for the liquefied condition by the drained p_{ult}). These equivalent p-multipliers were also calculated as they provide a convenient comparison to prior studies and are convenient for use with the LPile$^+$ program.

Typical analysis results are shown in Figures 12 and 13 for the 0.73-m diameter single pile. These two figures show results for the case (a) and case (e) soil displacement profiles, respectively. The best agreement between calculated and measured values for the bending moments and pile displacements was obtained for the case (e) soil displacement profile, which would be expected since case (e) most closely approximates the observed soil displacement profile. In this case, the calculated peak bending moment, which occurs just below the bottom of the loose sand layer, was about 21% larger than the measured value. For the case (a) soil displacement profile, the calculated peak bending moment was about 42% larger than the measured value. Calculated pile displacements were slightly larger for case (a) than for case (e), but both are in reasonable agreement with the measured pile displacements.

The results of the parametric study on soil displacement profiles (cases a-e) are summarized in Figure 14 for all the piles in the first centrifuge test. The best overall agreement was obtained with case (e) soil displacements, as previously noted for the single middle-sized pile. For case (e), bending moments in the elastic piles were predicted within -10% to +20%. For cases (a-d), the bending moments in the elastic piles were predicted within about –10% to +60%. Yielding of the SP pile was correctly predicted, but the magnitude of curvature was over-predicted by 45-150%, and the distribution of plastic yielding was not well captured. Lastly, it is worth noting that having four different pile systems in the same soil profile provided a more robust test of the design method because the same parameters had to be used for all four piles. Variations in certain input parameters were able to produce even better agreement for any individual pile, but not necessarily in all piles simultaneously. Hence, comparisons against the four piles gave a better sense of the repeatable accuracy of the design method.

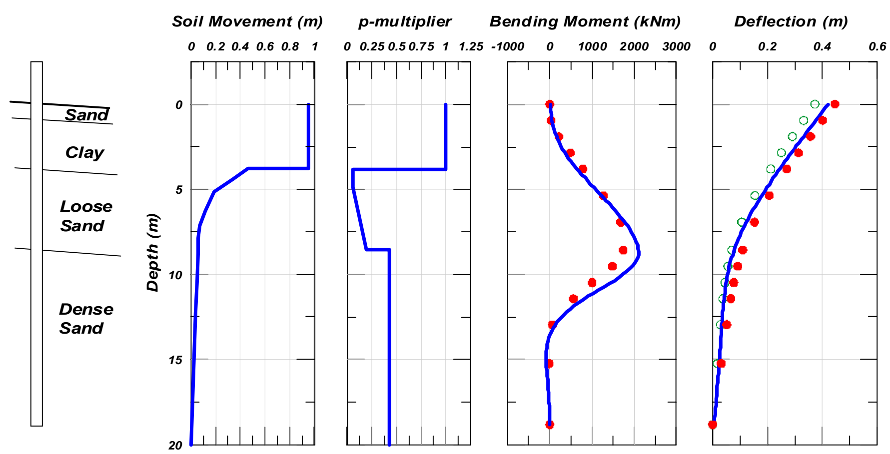


Depth (m)	C_u(kPa)	?'	K (MN/m^3)
0 - 0.95	--	33°	27
0.95 - 3.81	23	--	--
3.81 - 8.57	--	30°	6
8.57 - 20.0	--	39°	30

Pipe Pile:
Diameter = 0.73 m
Length (below Ground) = 18.8 m
EI = 308 MNm²
M_{ult} = 3.52 MNm

Peak Values
End-of-Shaking Values
Analysis: Case (a)

Fig. 12. Calculated and measured responses of the single middle-sized pile for case (a).



Depth (m)	C_u(kPa)	?'	K (MN/m^3)
0 - 0.95	--	33°	27
0.95 - 3.81	23	--	--
3.81 - 8.57	--	30°	6
8.57 - 20.0	--	39°	30

Pipe Pile:
Diameter = 0.73 m
Length (below Ground) = 18.8 m
EI = 308 MNm²
M_{ult} = 3.52 MNm

Peak Values
End-of-Shaking Values
Analysis: Case (e)

Fig. 13. Calculated and measured responses of the single middle-sized pile for case (e).

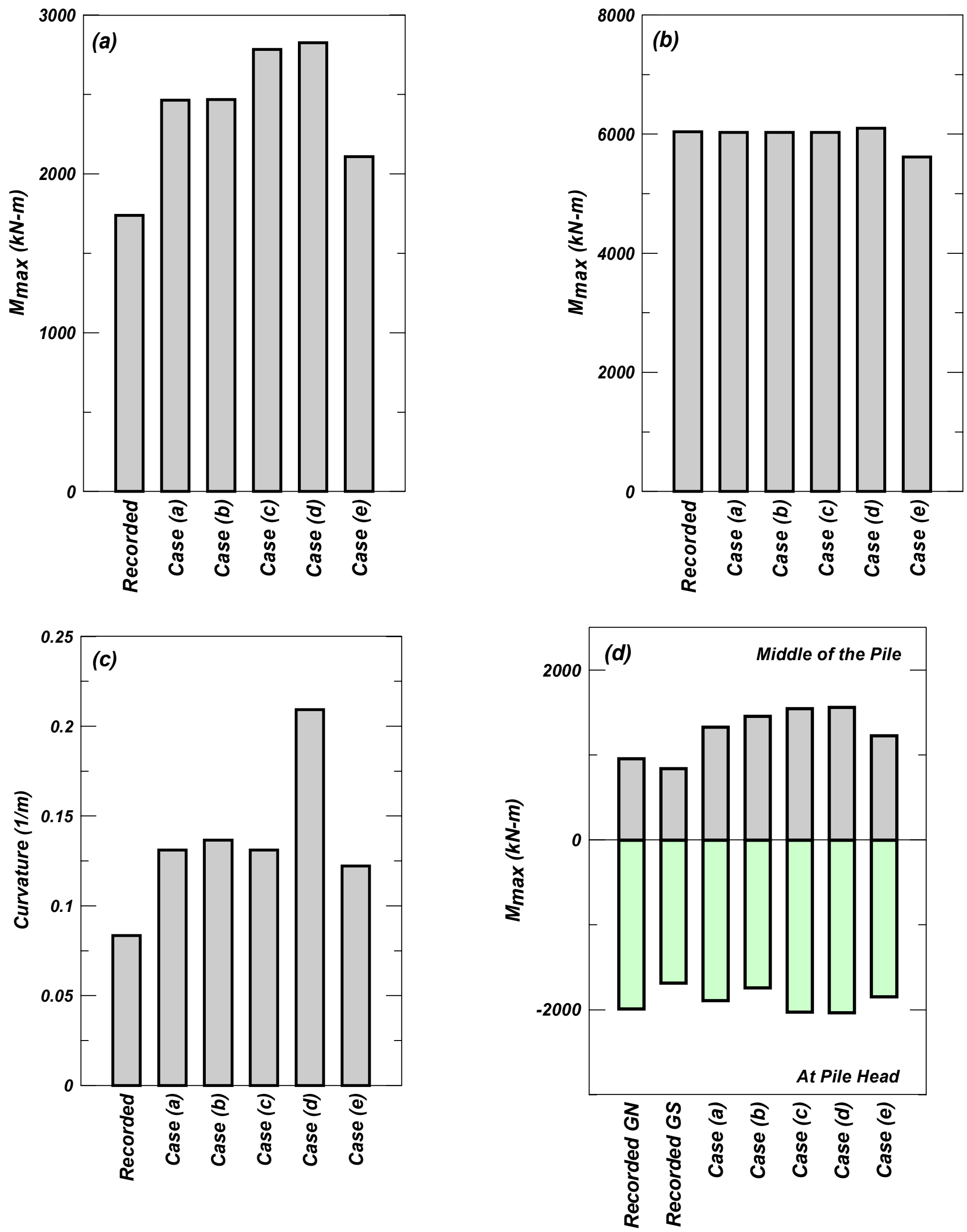


Fig. 14. Comparison of calculated and recorded maximum moments: (a) MP, (b) BP, (c) SP, (d) GP

DESIGN CONSIDERATIONS & CONCLUDING COMMENTS

Kinematic loading on piles in liquefying and laterally spreading ground during earthquakes can be a governing load case for design or retrofit studies. In recent years, case history studies have provided many valuable lessons and physical modeling studies have provided insights into the mechanisms of interaction. The present study has involved obtaining experimental data by dynamic centrifuge modeling to better understand the mechanisms of interaction and to evaluate simplified pushover analysis methods for design practice.

The p-y behavior of liquefying sand is only crudely approximated in simplified pushover analyses, and thus involves considerable uncertainty. Different investigations have resulted in considerably different estimates of the p-y behavior for liquefied soils, but these differences can be largely understood based on the known monotonic and cyclic loading behavior of saturated sands. The importance of the p-y approximation for liquefied soil depends on the particular problem. For example, there are cases where a nonliquefied crust can strongly dominate the lateral loads imposed on a pile foundation, rendering the calculated response 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. Our recommendation is to consider the first-order effects of relative density and cyclic loading condition when estimating the p-y behavior of liquefied soil.

Simplified pushover analyses of the centrifuge data presented herein generally provided reasonable to conservative estimates of bending moment and displacement demands. These monotonic analyses did not capture the effects of cyclic ratcheting on permanent displacements (rotations) for the pile groups. Analysis results were most sensitive to the shear strength of the nonliquefied crust, the soil displacement profile, and the compatibility of the soil displacement profile with the p-y approximations.

Assuming softer or weaker soil properties for p-y relations is generally (but not exclusively) unconservative for pushover analyses of lateral spreading problems. Instead, it is appropriate to use the best possible estimates of p-y resistance and then allow for appropriate uncertainty (stronger or weaker) in the design process.

Conventional cyclic loading factors in p-y relations for clay or sand (nonliquefied) should not be used in pushover analyses for lateral spreading. Such cyclic loading factors were derived for very different loading conditions, and their use will normally result in an underestimate of the loads imposed on piles by laterally spreading ground.

Pushover design analyses should consider different combinations of loading conditions, allowing for uncertainty in the main input parameters (soil displacement profile, p-y relations, etc.). It is not easy to always know whether a high or low parameter value will be conservative. In fact, for many parameters, it is possible to identify cases where a high value would have been conservative and other cases where a low value would have been conservative. Consequently, it is often misleading to generalize the results of a sensitivity study from one situation to another.

While this study is concerned with lateral spreading loads, it should also be recognized that significant down-drag load (negative skin friction) may develop on piles, particularly when an overlying nonliquefied layer settles due to re-consolidation of an underlying liquefied layer. Significant down-drag loads were observed in the centrifuge experiments with pile groups.

Nonlinear dynamic FEM analyses have certain advantages over the simplified pushover analyses, and their use in design may be advantageous under certain conditions. For example, dynamic FEM analyses may provide improved insight and guidance on how to address the following issues: the accumulation of pile foundation deformations due to ratcheting behavior during each cycle of loading, the appropriate combination of inertial and kinematic loads, the effect of progressive softening during shaking on the inertial and kinematic loads, and the effect of the substructure on the free-field ground deformations.

Lastly, the experimental data from this research project has been documented, archived, and made available on the web for use by other researchers. Interested individuals can access the data from the web site for the Center for Geotechnical Modeling, *http://cgm.engr.ucdavis.edu/.*

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