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Publication Date

2024-01-20

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International Symposium on Earthquake Engineering Commemorating
Tenth Anniversary of the 1995 Kobe Earthquake (ISEE Kobe 2005)
Kobe and Awaji, January 13 - 16, 2005

MECHANISMS OF LOAD TRANSFER BETWEEN PILE GROUPS AND LATERALLY SPREADING NONLIQUEFIED CRUST LAYERS

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SUMMARY

Observations of pile damage from the 1995 Kobe earthquake provided clear illustrations of the importance of loads imposed by laterally spreading ground. Analyses of these and other case histories have shown that the loads of greatest concern are often those imposed by nonliquefied surface (crust) layers that spread laterally over a liquefied layer. To study the mechanisms of load transfer between pile groups and laterally spreading nonliquefied crust layers, a series of dynamic centrifuge model tests were performed on a 9-m-radius centrifuge. Pile groups consisting of 6 piles, with prototype diameters of either 0.73 or 1.17 m, connected together by a pile cap were embedded in soil profiles that consisted of gently sloping nonliquefied crusts over liquefiable loose sand over dense sand. Models were shaken with earthquake motions having peak base accelerations from 0.13 g to 1.00 g. Time series for the soil-pile and soil-pile-cap loads were either measured directly or determined by back-calculation from the dense instrumentation arrays. The relative displacements between the free-field soil and pile cap that were required to mobilize the peak horizontal loads from the crust were much larger than expected based on analogies to static loading conditions. The mechanisms responsible for this softer-than-expected lateral load transfer behavior are explained and a simple model for describing the observed lateral load transfer behavior between a pile group and nonliquefied crust is developed.

1. INTRODUCTION

Loads from laterally spreading ground have been a major cause of damages to pile foundations in past earthquakes, including the 1995 Kobe earthquake (e.g., Tokimatsu et al. 1996, Karube and Kimura 1996, Hamada and Wakamatsu 1996, Matsui and Oda 1996). Analyses of these case histories have shown that the loads of greatest concern are often those imposed by nonliquefied surface (crust) layers that are spreading laterally over an underlying liquefied layer.

This paper describes the lateral load transfer behavior between pile groups and laterally spreading soil during earthquake shaking based on a series of dynamic centrifuge model tests. The centrifuge models involved nonliquefied surface layers of clay spreading laterally against pile groups. The observed lateral load transfer behavior is shown to be much softer than expected based on current design practice. A simple model is developed that describes the mechanisms by which liquefaction causes the lateral load transfer behavior from the clay crust to be softened relative to the static load transfer behavior in non-liquefied ground. This simple model is used to illustrate how other factors, beyond those covered in the centrifuge tests, are expected to affect the lateral load transfer behavior. Implications of these findings for pile foundation design are discussed.

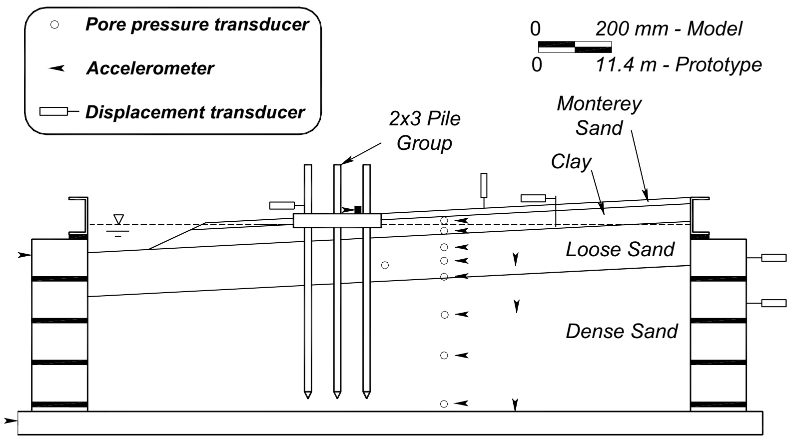

Figure 1. Schematic layout of a centrifuge model with a pile group in laterally spreading ground.

2. CENTRIFUGE MODELS WITH LATERALLY SPREADING CRUST LAYERS

Centrifuge models involving pile groups and laterally spreading crust layers, as schematically depicted in Figure 1, were performed in a flexible shear beam container on a 9-m radius centrifuge at centrifugal accelerations of 36 to 57 g. Results are presented in prototype units unless otherwise noted. The soil profile consisted of a nonliquefied crust overlying loose sand ($D_r \approx$ 21-35%) overlying dense sand ($D_r \approx$ 69-83%), as shown in Figure 1. The crust layer sloped gently toward a river channel carved in the crust at one end of the model. The nonliquefiable crust consisted of reconstituted San Francisco Bay mud (liquid limit ≈ 88, plasticity index ≈ 48) that was mechanically consolidated with a large hydraulic press, and subsequently carved to the desired slope. The sand layers beneath the crust consisted of uniformly graded Nevada Sand (C_u = 1.5, D_{50} = 0.15 mm). A thin layer of coarse Monterey sand was placed on the surface of the Bay mud for some of the models.

The six-pile group for the model in Figure 1 consisted of 1.17-m diameter piles with a large pile cap embedded in the nonliquefied crust. The pile cap provided nearly fixed-head restraint at the connection with the piles.

Each test was shaken with a number of simulated earthquakes conducted in series with sufficient time between shakes to allow dissipation of excess pore pressures. Generally, the shake sequence applied to the models was a small event ($a_{max,base}$ = 0.13g to 0.17g) followed by a medium event ($a_{max,base}$ = 0.30g to 0.45g) followed by one or more large events ($a_{max,base}$ = 0.67g to 1.00g). Details for these centrifuge experiments are summarized in a series of data reports available from the web site for the Center for Geotechnical Modeling (http://cgm.engr.ucdavis.edu) (e.g. Brandenberg et al. 2003).

3. OBSERVED LOAD TRANSFER BEHAVIOR IN CENTRIFUGE MODELS

The total lateral loads on the pile groups during earthquake shaking were determined from both shear gages and differentiation of bending moment distributions along the piles (Boulanger et al. 2003, Brandenberg et al. 2004). The lateral load from the clay crust (F_{crust}) was then computed as the difference between the total lateral load and the inertial load of the pile cap and any structure connected to the cap.

The total lateral crust load is plotted versus the relative displacement at virgin loading peaks (i.e. crust load peaks that exceed the maximum past crust load) in Figure 2. Each virgin peak load was normalized by the greatest overall peak load measured for that specific model test. The relative displacement was taken as the crust surface displacement to the side of the pile cap minus the pile cap displacement. This relative displacement was then normalized by the thickness of the nonliquefiable crust layer. The resulting data show that the overall peak lateral loads were mobilized at relative displacements of 25% to over 60% of the pile cap height, which is much larger than commonly expected. Brandenberg et al. (2004) also presented a simple equation for approximating the observed behavior, with the computed relation shown in Figure 2 for four different values of the primary fitting parameter, C. Note that the choice of C = 0.04 produced a load transfer relation that is similar to relations used for static loading tests.

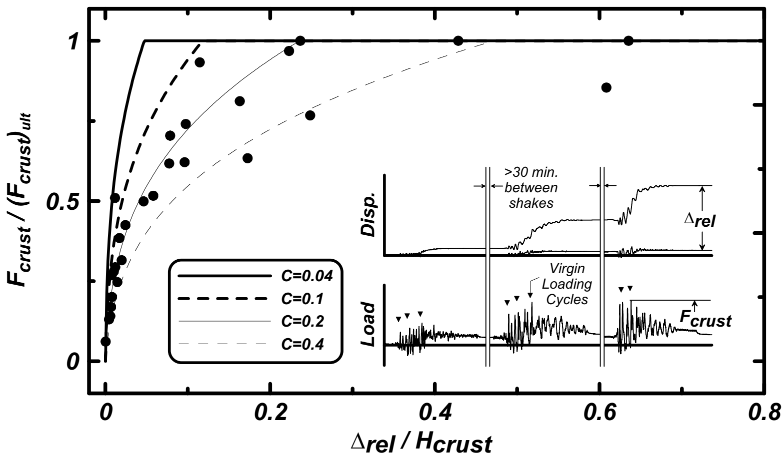

Figure 2. Normalized lateral load from the surface crust versus relative cap-soil displacement

The observed load transfer data shown in Figure 2 is much softer than expected based on common experiences with static loading tests. Static loading of retaining walls and pile caps have shown that full passive resistance is mobilized when the wall displacement is more than about 1% to 5% of the wall height, depending on soil type and density. For example, Rollins and Sparks (2002) performed static load tests on a pile group in granular soil and reported that the peak load was mobilized at a pile cap displacement of about 2.5% to 6% of the pile cap height. Duncan and Mokwa (2001) and Mokwa and Duncan (2001) describe load tests on bulkheads and pile groups embedded in sandy silt/sandy clay and in gravel/sand backfills and showed that passive loads were mobilized at displacements of about 1% to 4% of the pile cap height. In design practice for lateral spreading loads, the load transfer relation is commonly assumed to be similar to those observed from static loading tests.

The observed deformation patterns in the centrifuge models (Brandenberg et al. 2004) showed that the pile group influenced the ground deformation pattern in the clay crust to large distances upslope of the pile group. Strains in the clay crust were greatest immediately upslope from the pile cap, and decreased gradually with distance upslope from the cap. Crust displacements near the sides of the pile cap were also smaller than the displacements near the walls of the container. Note that the friction between the crust and container walls was minimized in some tests by cutting a thin slot through the crust along its contact with the container wall, and then injecting bentonite slurry into the slot prior to testing of the model. The large zone of influence around the pile group will later be shown to be directly related to the observed lateral load transfer relation in Figure 2.

The lateral load from the clay crust includes loads on the pile cap and the pile segments that are within the crust. The ultimate or peak lateral crust load $(F_{crust})_{ult}$ may be controlled by either of two mechanisms, as illustrated in Figure 3. One possible failure mechanism (Figure 3a) is for the clay to flow around the pile segments, such that the crust load is the sum of the ultimate tractions on the pile segments (i.e., p_{ult}) and the ultimate tractions on the pile cap (sum of passive forces on the upslope face and friction along the sides and possibly the base of the pile cap). The second possible mechanism (Figure 3b) is where the clay does not flow around the pile segments, but rather the pile group acts as a larger block that develops passive loads throughout the clay layer thickness on the upslope side, along with friction along the sides. The governing failure mechanism would be the one that produces the smallest overall load against the pile group. Post-testing photographs of the excavated models (Brandenberg et al. 2004) indicate that the first mechanism (Figure 3a) governed in the centrifuge tests described herein. Additional studies are required to clarify how liquefaction beneath the crust influenced the load transfer behaviors for the pile segments alone and the pile cap alone, and how it would influence the load transfer behavior when the pile cap and segments act as a combined block.

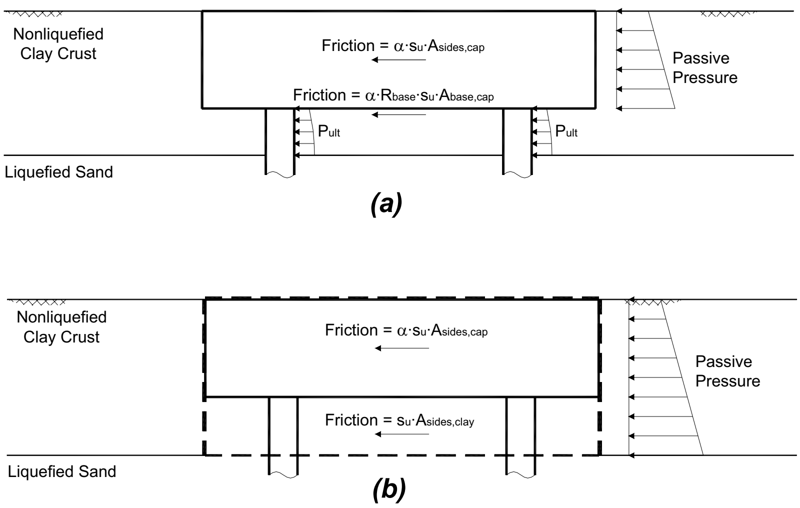

Figure 3. Schematic of lateral loads on pile group when (a) the crust spreads around the individual piles and (b) when the crust is trapped between the piles and the group behaves as a block.

4. MECHANISM OF LOADING FROM LATERALLY SPREADING CRUST

The softer-than-expected lateral load transfer behavior between the pile group and clay crust (Figure 2) is attributed primarily to two major influencing factors:

- Cyclic degradation of the clay stress-strain response and clay-pile-cap interface friction, which would cause the envelope of the cyclic lateral load transfer behavior to be softer than for static lateral loading.
- Liquefaction of the underlying sand, which would enable the loads from the pile cap to spread farther in the clay crust, thereby inducing significant stresses and strains over a larger zone around the pile group.

The distributions of stresses that develop in the crust around the pile cap are strongly affected by the occurrence of liquefaction in the underlying layer. For static loading of a pile cap without any liquefaction in the underlying soils, some of the stress imposed on the clay by the pile cap would geometrically spread down into the sand and thus stresses in the clay crust would decrease sharply with distance away from the pile cap. For the case where the underlying sand is liquefied, the stress imposed on the clay by the pile cap would not be able to spread down into the liquefied sand (assuming it has essentially zero stiffness compared to the crust), and thus the lateral stress in the clay crust would decrease more slowly with distance away from the pile cap.

The deformation pattern that develops in the crust around a pile group is schematically illustrated in Figure 4. Ground displacements at large distances from the pile group are unaffected by the presence of the pile group, and are thus call free-field displacements. Ground displacements on the upslope side and near to the pile group are less than their corresponding free-field values because of the restraining forces from the pile group. The relative displacement between the pile group and the free-field soil ground displacements represent an integral of the change in horizontal strain induced by the foundation between the pile group and some free-field reference point. This relative displacement is an important measure of the lateral load transfer behavior because it is the free-field soil displacement that is usually estimated and input as a loading condition for the design of a pile foundation.

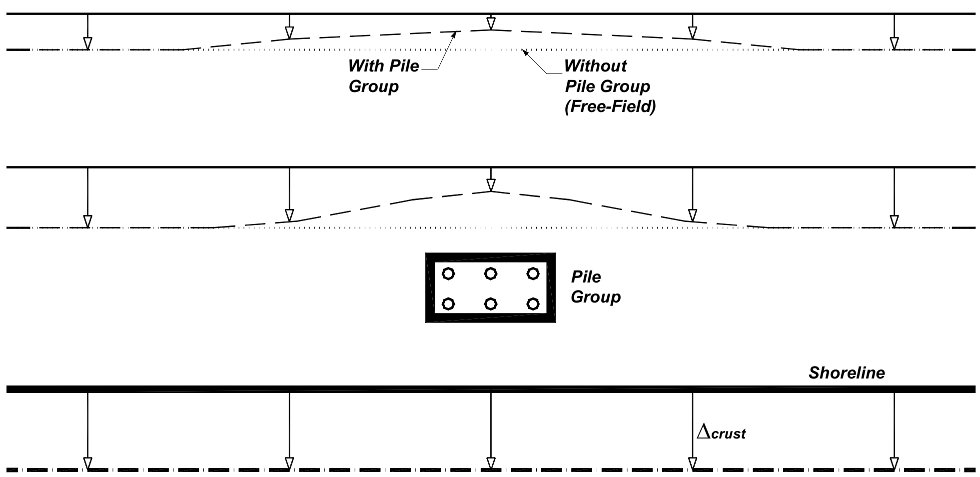

Figure 4. Plan view schematic of the influence of a pile group on the lateral displacements of a nonliquefiable layer spreading on top of a liquefied layer.

4.1 Two-dimensional Model of Lateral Load Transfer

A simple two-dimensional idealization of the lateral loading mechanism provides a clear illustration of the basic mechanisms involved in making the lateral load transfer behavior softer (based on a relative displacement between the pile group and the free-field ground surface). Consider the lateral loading of the clay layer over liquefied soil as idealized in Figure 5. The clay layer is H thick, L long, and its material behavior is idealized as elastic-plastic with a yield strain of ε_{hf} = 5% (or an ε_{50} of 2.5%, where ε_{50} is the strain when the deviator stress is 50% of the ultimate deviator stress in a triaxial compression test). The bottom boundary of the clay rests on liquefied sand that has a specified value of residual shear strength (s_r). The vertical boundaries of the clay are both frictionless, and the right boundary is fixed against translation.

The lateral load-displacement behavior for the wall in Figure 5 is first illustrated for the case where $s_r = 0$ (e.g., a perfect water film forms beneath the clay layer). In this case, the lateral stresses and strains in the clay crust do not change with distance from the wall, but rather remain constant throughout the clay crust. The peak passive lateral force will be reached when the lateral displacement, Δ_{hf}, is:

$$\Delta_{hf} = \varepsilon_{hf} \cdot L \tag{1}$$

This lateral displacement can be normalized by the wall height to obtain:

$$\frac{\Delta_{hf}}{H} = \frac{\varepsilon_{hf} \cdot L}{H} \tag{2}$$

Equations (1) and (2) were used to produce the plot of normalized lateral load versus normalized displacement in Figure 6, where P is the load per unit width of wall. When the clay layer length is equal to the wall height (2.5 m), the peak lateral load is mobilized at a normalized displacement of Δ_{hf}/H = 0.05. In contrast, when the clay layer length is 40 m, the peak lateral load is mobilized at a normalized displacement of Δ_{hf}/H = 0.8.

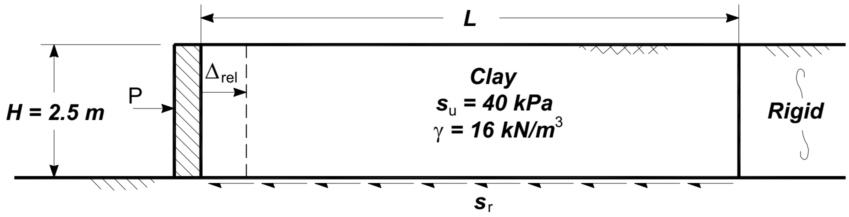

Figure 5. Two-dimensional idealization for illustrating the effect of liquefaction on load transfer between a pile cap and a nonliquefied crust overlying liquefied soil.

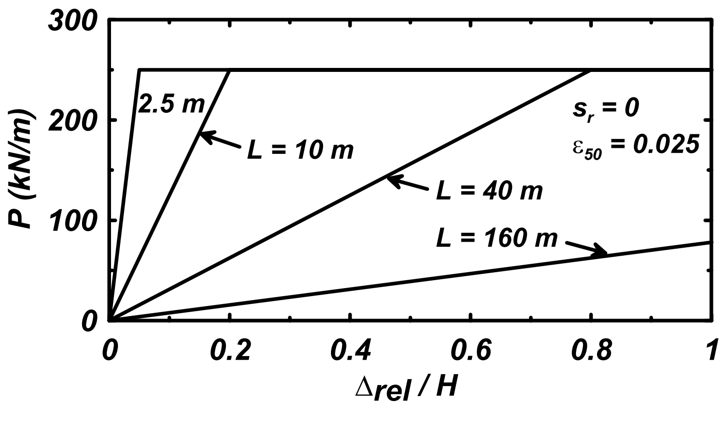

Figure 6. Lateral load transfer relations for idealized 2-D problem with elastic-plastic clay layer of various lengths overlaying a liquefied soil with zero shear resistance.

The influence of s_r on the load-displacement behavior for the two-dimensional wall in Figure 5 is next illustrated for the case where the crust length is very large. At any distance x from the wall, the lateral stress in the crust is computed from the load on the wall minus the load resisted by the liquefied soil to that distance. In this manner, the lateral stress in the crust decreases with distance from the wall, until it reaches zero at a distance L_{cr}. The displacement of the wall is then the integral of the strains in the crust over this distance L_{cr}. The computed load-displacement behavior for s_r values of 2, 4, and 8 kPa is shown in Figure 7, and the variation in L_{cr} with s_r is shown in Figure 8. Note that s_r values of only a few kPa are consistent with the observed crust displacements based on Newmark sliding block analyses of the clay crust. Figures 7 and 8 illustrate how an increase in shear resistance in the liquefied sand causes the zone of influence (L_{cr}) to decrease and the load-displacement behavior to stiffen (i.e., the relative displacement at a given load level decreases).

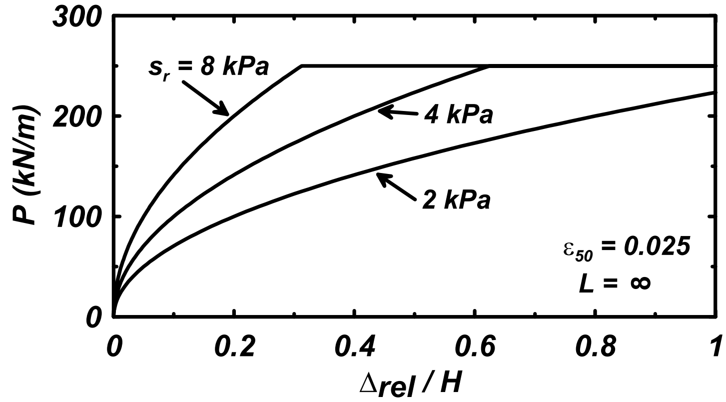

Figure 7. Lateral load transfer relations for idealized 2-D problem with infinitely long elastic-plastic clay layer over liquefied sand layer with various residual strengths.

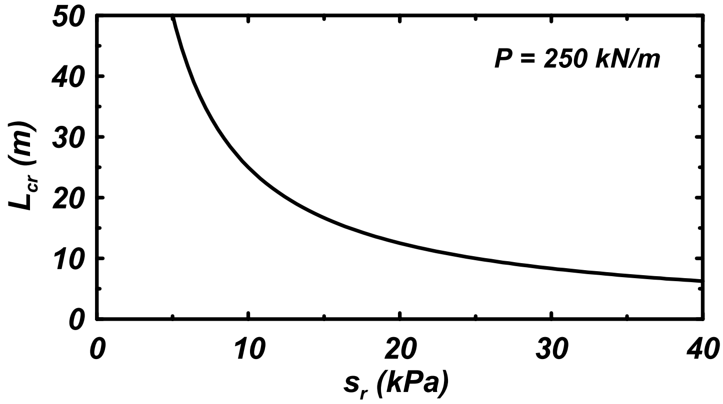

Figure 8. Critical length defining the size of the zone of influence upslope of the pile cap versus the residual shear strength of the liquefied layer.

The computed zone of influence ahead of the wall reduces to only a few meters (or about twice the wall height) when the residual shear strength of the liquefied soil is equal to the undrained shear strength of the clay crust (Figure 8). In this case, the ultimate lateral load would occur at a wall displacement equal to only a few percent of the wall height, similar to that expected for a static loading case. When the residual shear strength of the liquefied layer is only a few kPa, however, the zone of influence extends to distances in excess of 20 to 40 m. These expected zones of influence are reasonably consistent with the observed patterns of ground deformation in the centrifuge models described by Brandenberg et al. (2004).

4.2 Three-dimensional Model of Load Transfer

The effect of three-dimensional loading conditions that occurs for pile caps of finite width was evaluated using a simple 2:1 stress distribution method for the clay crust. In plan view, the load on the pile cap face was assumed to spread at a 2:1 ratio in the upslope direction (i.e., if the pile cap width was W, then the equivalent loaded area would be "W+x" wide at a distance x from the pile cap). The lateral stress at a distance x from the pile cap would now reduce both due to the increase in the equivalent loaded area and due to the resistance provided by the underlying liquefied soil. The load-displacement behavior for a pile cap width to height ratio of 4 is compared to that for the plane strain case, both with s_r = 4 kPa, in Figure 9. This example illustrates how the load-displacement behavior is expected to be softer with increasing W/H ratios.

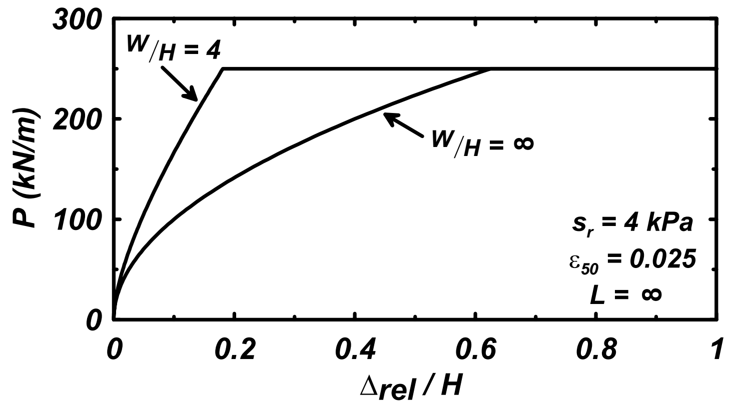

Figure 9. Lateral load transfer relations for infinitely long elastic-plastic clay layer over liquefied sand assuming that the lateral stresses attenuate at 2:1 upslope from the pile cap for various pile cap width to height ratios.

4.3 Discussion of Load Transfer Models and Results

The effects of cyclic degradation on the load transfer behavior are only roughly accounted for in the preceding analyses by the selection of an appropriately softened secant stiffness (or ε_{50} value). The actual mechanisms of cyclic degradation in the load transfer behavior will include the cyclic degradation of the clay plus cyclic degradation of the soil-pile and soil-pile-cap interfaces and the effects of crust cracking and interface gapping. In this manner, the computed load transfer relation is intended to approximately envelope the cyclic load versus relative displacement behavior. The appropriate choice of ε_{50} values will consequently depend on the soil characteristics and the nature of the ground motions, as well as on other factors.

The dynamic loading conditions between a pile group and nonliquefied crust are more complicated than accounted for in the simple pseudo-static load transfer model developed herein. This simple model is only intended as an idealization for the increment of loading that arises from the crust-pile-group interaction; for example, the actual shear stresses along the base of the crust would clearly depend on the dynamic site response and thus vary in time. Despite these simplifications, the load-displacement relations predicted by this simplified model are reasonably consistent with the load transfer behavior observed in the centrifuge model tests, which were previously summarized in Figure 2. In particular, the simple model illustrates how the occurrence of liquefaction beneath the crust layer affects the distribution of stresses in the clay crust, the zone of influence around the clay crust, and hence the relative displacements that develop between the pile cap and the free-field ground surface at a given level of lateral loading.

The analysis of lateral load transfer behavior needs to be modified for situations where the extent or mass of the nonliquefied surface layer is small enough that it is all within the zone of influence of the pile group's resisting forces (i.e., there is no free-field condition for the surrounding ground). For example, consider an earthen bridge abutment whose lateral spreading is restrained by piles across its full width. The restraining force from the piles can reduce the amount of lateral spreading that the abutment develops, such that the lateral force that develops against the piles never rises to the passive capacities of the soil (e.g., Martin et al. 2002). The analysis of such a situation requires computing the displacement of the soil abutment and the pile foundation, and may include pushover analyses of the pile foundations to define their interaction. The relations proposed herein may not be appropriate in such cases because the displacement of the abutment can not be considered "free-field", and such pushover analyses would need to use a load transfer relation that is based on the relative displacement between the piles and the adjoining abutment soils.

5. SUMMARY

Insight on the load transfer relations between pile groups and laterally spreading crust layers was obtained from a series of dynamic centrifuge model tests with pile groups embedded in a sloping soil profile with nonliquefied clay overlying liquefiable loose sand over dense sand. The nonliquefied crust laterally spread downslope on top of the liquefiable sand, imposing large kinematic loads on the pile groups. The experimental data showed that large relative displacements between the pile caps and the free-field soil were required to mobilize the peak lateral crust loads against the pile caps. This lateral load transfer behavior was much softer than expected based on analogies to static loading experiences.

The relatively soft lateral load transfer behavior that was observed in these experiments is attributed primarily to the effects of cyclic degradation and the influence of liquefaction on the stress distributions with the crust layer. Liquefaction beneath the crust layer results in a very low shear resistance along the base of the crust. The reaction forces from the pile group can therefore cause stresses in the crust to spread geometrically to larger distances upslope of the pile cap than would occur if the underlying soil was not liquefied. The resulting distribution of stresses and strains over a larger zone of influence resulted in larger relative displacements because relative displacement is the integral of strain between two reference points.

Simple two- and three-dimensional models for the lateral load transfer behavior between pile groups and laterally spreading nonliquefied crust layers were developed. Despite the simplifications involved in these models, the predicted load-displacement relations appear to provide a reasonable means for explaining the load transfer behavior observed in the centrifuge model tests.

6. ACKNOWLEDGEMENTS

Funding was provided by the Pacific Earthquake Engineering Research (PEER) Center, through the Earthquake Engineering Research Centers Program of the National Science Foundation, under contract 2312001, and by Caltrans under contract numbers 59A0162 and 59A0392. The contents of this paper do not necessarily represent a policy of either agency or endorsement by the state or federal government. Recent upgrades to the centrifuge have been funded by NSF award CMS-0086566 through the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES).

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