EXPERIMENTAL P-Y CURVES FROM CENTRIFUGE TESTS ON PILE FOUNDATIONS SUBJECTED TO LIQUEFACTION AND LATERAL SPREADING

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ABSTRACT

The results of five centrifuge models were used to evaluate the response of pile-supported wharves subjected to inertial and liquefaction-induced lateral spreading loads. The centrifuge models contained pile groups that were embedded in rockfill dikes over layers of loose to dense sand and were shaken by a series of ground motions. The p-y curves were back-calculated for both dynamic and static loading from centrifuge data and were compared against commonly used API p-y relationships. It was found that a significant reduction in ultimate soil resistance occurred in dynamic p-y curves in partially/fully liquefied soils as compared to static p-y curves. It was also found that incorporating *p*-multipliers that are proportional to the pore water pressure ratio in granular materials is adequate for estimating pile demands in pseudo static analysis.

Keywords: Pile foundations, Liquefaction, Lateral spreading, Centrifuge models, p-y curves.

1. INTRODUCTION

Liquefaction-induced ground deformations can cause severe damage to pile-supported wharves and other waterfront structures. A common approach in analyzing the lateral behavior of piles against seismic loads is using the beam on nonlinear Winkler foundation (BNWF) or p-y spring analysis. One common p-y relationship for sand is the one proposed by the American Petroleum Institute, also known as the API sand model (API 1993). While the API sand model was originally developed for the static loading conditions, it is common to modify the API sand curves for the effects of cyclic loading. However, there is no consensus on how to modify the static p-y curves for the effects of liquefaction and pore water pressure generation in loose granular soils. In previous studies, p-y springs of piles in liquefying soils were back-calculated from case histories, centrifuge model studies (e.g., Wilson et al. 2000; Brandenberg et al. 2005; Abdoun et al. 2003), full-scale tests (e.g., Rollins et al. 2005; Chang and Hutchinson 2013), and numerical analyses (e.g., McGann et al. 2011). In general, these studies found the softening effects of soil liquefaction on p-y curves, as well as a hardening behavior attributed to the dilative response of the liquefied soil around the pile. However, these studies provide varying recommendations on how to modify the static p-y curves to capture this complex behavior. Some design guidelines propose softening the static p-y curves using *p*-multipliers (e.g., Caltrans 2012). Other studies propose an upward concave shape for p-y curves in liquefied soils (e.g., Franke and Rollins 2013; Chang and Hutchinson 2013).

The focus of this study is to evaluate the effectiveness of the *p*-multiplier approach in modifying p-y springs in partially/fully liquefied soils to predict the lateral response of piles. This was done by using the results of five centrifuge tests simulating pile-supported wharves in sloping ground (McCullough et al., 2001). The p-y curves were back-calculated in loose sands, dense sands and sloping rockfill dikes. The p-y curves were back-calculated for both static piles subjected to push/pull forces at pile head, as well as dynamic piles subjected to earthquake shaking. The static p-y curves were approximated using API relationships for sands and the input parameters for API curves were back-calculated. The dynamic p-y curves were compared against the static p-y curves to provide insights on the applicability of the *p*-multiplier approach in developing p-y curves for liquefied zones. To evaluate the effectiveness of using *p*-multipliers on API sand curves, the piles in centrifuge tests were modeled using pseudo-static analysis in *LPILE* (Ensoft 2014) and the predicted maximum bending moments in each pile were compared against the measured values in centrifuge tests. It will be shown that the maximum bending demands in piles were reasonably captured using p-multipliers that are proportional to the pore water pressure ratio in partially/fully liquefied zones.

2. DESCRIPTION OF CENTRIFUGE TESTS

2.1. Centrifuge models and cross sections

Data from a series of five centrifuge tests were analyzed to back-calculate pile lateral behavior (i.e. p-y springs) for static and dynamic loading conditions. These tests were performed on pile-supported wharves by Dickenson, McCullough, Schlechter, and coworkers at the UC Davis Center for Geotechnical Modeling (McCullough et al. 2001). These centrifuge models represent the typical layout of major port facilities in California, and the findings can be used to represent other similar pile-supported wharves embedded in rock dikes over native, potentially liquefiable soils. The cross sections of all models and key soil properties are shown in Figure 1.

2.2. Dynamic piles

The wharf deck was supported by three rows of seven piles (for a total of 21 piles). The pile diameters ranged from 0.38 m to 0.68 m. Each centrifuge model was subjected to a sequence of scaled input motions with the peak base acceleration ranging from 0.15 g to 0.82 g. The piles were subjected to the combined effects of inertial and liquefaction-induced kinematic demands during earthquake shaking (these piles are referred to as *dynamic piles*). The dimensions discussed in this paper are all in prototype scale unless noted otherwise.

2.3. Static piles

Two of the five tests (SMS02 and JCB01) included two single piles that were statically pushed by two to seven cycles of loads using actuators attached to their pile heads (these piles are referred to as *static piles*). The static loads, which were applied prior to earthquake shaking, provided key data for comparing the p-y springs under static and dynamic loading conditions. In these two tests, the static pile at the back was placed in dense sand with no slope; the static pile at the front was placed in sloping rockfill in SMS02 and in a sloping rock face overlying loose sand in JCB01. The layout for the static piles is shown in Figure 1. The properties of the static piles were the same as those for the dynamic piles.

2.4. Sensors and instruments

Measurements for all centrifuge tests conducted in this study were obtained using the following sensors and instrumentation. Linear volt displacement transducers (LVDT) mounted on the wharf deck, ground surface and the shear box container were used to measure the horizontal and vertical displacements. Pore pressure transducers (PPT) were embedded within the soil model at various depths. Accelerometers were embedded within the soil model and attached to the wharf deck and the shear box. Strain gauges were attached to static and dynamic piles to measure the bending moments.

3. PROCEDURES TO BACK-CALCULATE P-Y CURVES

3.1. *Lateral soil reactions*

Bending moments were measured at discrete locations along the pile where strain gauges were attached. The bending moments were interpolated along the pile length using the cubic spline fitting method before being numerically double-differentiated to back-calculate the lateral soil reactions, *p* (Haiderali and Madabhushi 2016; Brandenberg et al. 2010). For the piles where the bending moment at the pile head was not measured, the bending moments were extrapolated assuming a constant shear force above the ground surface. The bending moments and shear forces at the pile tips were assumed to be zero.

3.2. Horizontal pile displacements

The horizontal pile displacements were estimated by double-integrating the bending moments along the pile and dividing them by the pile flexural stiffness (*EI*). The rotations at the pile head were assumed to be zero as the piles were rigidly connected to a relatively rigid wharf deck.

3.3. Horizontal soil displacements

Total horizontal soil displacements were calculated by combining the transient (high-frequency) and permanent (low-frequency) components of displacement following methods described by Wilson et al. (2000). The transient soil displacements were calculated by double-integrating the recorded accelerations. A high-pass Butterworth filter was applied to remove the low-frequency motions from the recorded accelerations. The permanent soil displacements were calculated based on the displacements recorded using LVDTs at the ground surface after applying a low-pass Butterworth filter. The pattern of distributing the permanent component of the soil displacement with depth was a major source of uncertainty in our analyses. The estimated pile bending moments in our consecutive pseudo-static analyses were also found to be very sensitive to the assumptions made regarding the pattern of permanent soil displacements with depth, which warranted investigating this issue methodically. After considering various patterns of permanent soil displacement with depth and investigating their effects on the estimated bending moments, we used the normalized shape of the maximum transient displacements with depth as a guide to determine where the subsurface shear failure zones formed as well as to distribute the permanent component of the soil displacement from the ground surface down to the shear failure plane. No permanent soil displacement was considered below the shear failure plane.

3.4. Back-calculated p-y curves

Lateral pile behavior is commonly characterized using p-y curves extracted at various depths along the pile. The p in these relationships corresponds to the lateral soil reaction, and the y corresponds to the relative displacement between the soil and pile (i.e. y = horizontal pile displacement – horizontal soil displacement). As described earlier, there is some uncertainty in estimating the horizontal soil displacements and pile displacements for dynamic piles. Therefore, the dynamic p-y curves were used primarily for estimating ultimate lateral soil reaction, and the relative soil–pile displacement (i.e. y) was only used qualitatively.

4. RESULTS

4.1. Experimental p-y curves from static piles

Experimental p-y curves were extracted from the results for statically loaded piles in SMS02 (penetrating dense sand and rockfill) and JCB01 (penetrating dense sand, loose sand and a thin rockfill) prior to shaking. Given that these soil and rockfill units are made from granular materials, the back-calculated p-y curves were approximated using API sand relationships. The API sand model recommends a hyperbolic tangent function to characterize the ultimate soil reaction (p_{ult}) and the initial stiffness (k_T).

In the API sand model, the ultimate lateral reaction (p_{ult}) increases with depth, pile diameter and internal friction angle. Internal friction angles of 33°, 37° and 45° were used to develop API curves for loose sand (D_R = 30%), dense sand (D_R = 70% to 80%) and rockfill, respectively. Comparing the ultimate resistances of the API curves with those of the back-calculated p-y curves at shallow depths, where the ultimate soil reaction was mobilized, confirmed that the API relationships estimated the ultimate resistance of the p-y curves reasonably well. As an example, a comparison between the experimental p-y curve and the API relationship for loose sand is shown in Figure 2a for the front pile in JCB01 at a depth of 3.05 m (~5D). This static pile was subjected to seven cycles of static loading. Different loading cycles are plotted with different colors on this figure to help understand the evolvement of p and y in the experimental p-y curve.

As can be noticed from this figure, the API sand curve using a friction angle of 33° captures the ultimate resistance of the experimental p-y curve reasonably well. Additionally, the sand p-y curve proposed by Reese et al. (1974) was plotted in Figure 2a which aligned reasonably well with the experimental p-y curve. Figure 2b shows the 6th cycle of the same experimental p-y curve compared against the same API curve manually shifted to the left for plotting purposes. This figure clearly shows that the reduced modulus of subgrade reaction better captures the stiffness of the experimental p-y curves at larger displacements. Similar conclusions could be made for dense sand and rockfill units.

The initial stiffness in the API sand curve (k_T) is the product of the depth below the ground surface and the modulus of the subgrade reaction (k). The initial stiffness was back-calculated from the experimental p-y curves at various depths and within different soil units. These initial stiffness values were divided by the corresponding depth to obtain the modulus of subgrade reaction (k) for loose sand, dense sand and rockfill. It was found that the moduli of subgrade reaction calculated from experimental p-y curves were significantly smaller than the values recommended by API. For example, the modulus of subgrade reaction for loose sand was calculated as 3500 kN/m^3, while API suggests a value of 16000 kN/m^3. This might be attributed to the limitations of centrifuge tests in simulating the in situ soil stiffness (i.e. freshly deposited sands in the centrifuge vs. aged sands in the field) and the pile driving conditions. This highlights the need to incorporate lower- and upper-bound multipliers in developing soil springs in design as recommended by standard ASCE COPRI 61-41.

Table 1 lists the input parameters to use for API sand curves based on the experimental p-y curves obtained from static piles. No significant difference was observed in the back-calculated subgrade reaction moduli between loose and dense sands; therefore, the same modulus is recommended for simplicity.

Table 1. Back-calculated input parameters for API sand p-y curves

Soil unit	Friction angle	Initial modulus of subgrade reaction, k (kN/m^3)
Loose sand (D_R = 30% to 40%)	33°	3500
Dense sand (D_R = 70% to 85%)	37°	3500
Rockfill	45°	5200

4.2. Experimental p-y curves from dynamic piles

Experimental p-y curves were also derived from centrifuge tests for piles supporting the wharf deck. These piles were subjected to wharf inertia during shaking, combined with varying magnitudes of ground deformation induced by partial/full liquefaction. These dynamic p-y curves were then compared to the static p-y curves to investigate the effects of excess pore water pressure in liquefiable soils on the lateral response of piles and p-y curves.

Figure 3a presents a comparison of static versus dynamic p-y curves for loose sand (D_R = 40%). The static p-y curve shown in this figure was derived from the front static pile in JCB01 (same as Figure 2.) The dynamic p-y curve was derived for Pile #3 in JCB01 during the first earthquake motion. Both static and dynamic p-y curves are extracted at the same depth (3.05 m below the ground surface) and normalized by the same pile diameter (0.64 m). For clarity, the dynamic p-y curve is plotted only for the time window shown in the time histories in Figures 3b and 3c. These time histories illustrate the lateral soil resistance (p), relative lateral displacement between soil and pile (y), and pore water pressure ratio (R_u) in the loose sand. It can be observed that as the excess pore water pressure ratio increases in the loose sand in sloping ground, lateral spreading occurs that exerts lateral loads on the pile. It is also observed that the lateral soil reaction (p) in liquefied soil exhibits sudden spikes in the downslope direction. Careful examination of the spikes in p reveals that they follow transient drops in R_u implying that they might be attributed to the dilative response of sand combined with an increase in the relative displacement between the soil and pile driven by the inertial demand from the wharf deck. However, the magnitude of spikes in p are not very large (i.e. less than 20% of P_{ult} of the static p-y curve) suggesting that a simple p-multiplier approach could be an effective choice to modify the static p-y curve to envelope the complex behavior of dynamic p-y curve in liquefied soil.

Overlapped on Figure 3a are two API sand curves that approximate the p-y responses under static and dynamic conditions. The API sand curve for the static condition is developed using the input parameters in Table 1. The experimental dynamic p-y behavior is complex and is affected by contraction and dilation of loose sand and the inertial demand from the superstructure during the earthquake loading. To fit within the envelope of the dynamic experimental p-y curve, the API sand curve was softened using p-multipliers (P_m) to account for the first-order effects of liquefaction. Other researchers have shown that P_m depends on the pore water pressure ratio (R_u) generated during shaking (e.g., Wilson et al. 2000; Brandenberg 2005). For simplicity, the p-multipliers in this study were calculated using $P_m = 1 - R_u$. It is observed from Figure 3c that an R_u of approximately 0.8 was developed in the loose sand. Therefore, a P_m of 0.2 was used to soften the API curve, as shown in Figure 3a.

5. VALIDATION AGAINST PILE DEMANDS

The effectiveness of the back-calculated input parameters for API sand curves and the R_u-proportional p-multipliers in liquefiable soils is investigated by comparing the pile bending moment profiles measured in the centrifuge tests to those estimated using p-y models in *LPILE* (Ensoft 2014). The *LPILE* models considered combined kinematic and inertial effects, in which the soil displacements were imposed to the end nodes of p-y springs and wharf inertia was imposed by a shear force at the pile head. The kinematic demands (i.e. soil displacements) and inertial demands (i.e. pile head shear) were directly calculated from the centrifuge tests at the exact time when the bending moments are at their peak values. The p-y curves were developed for each soil unit based on the API relationships with the input parameters reported in Table 1. The p-y curves were then softened using p-multipliers proportional to the pore water pressure ratio (R_u) using the equation $P_m = 1 - R_u$. The R_u values were extracted from the pore pressure transducers (PPT) in the corresponding unit.

Figure 4a shows a snapshot of soil displacement profiles along two instrumented piles in JCB01 along with the pile displacement estimated from *LPILE*. Figure 4b shows a comparison of the bending moments obtained from centrifuge tests to those estimated from the *LPILE* analyses. The reasonable agreement between the measured and predicted pile bending moments validates the effectiveness of the API sand curves with the input parameters recommended in Table 1 and modified by p-multipliers that are proportional to the pore water pressure ratio (R_u) in granular materials. In this test, the recorded R_u values ranged between 0.9 and 0.5 at various depths in the loose sand and was approximately 0.4 in the dense sand. This corresponded to P_m values of 0.1 to 0.5 for the loose sand and 0.6 for the dense sand.

In order to further investigate the applicability of the modified API curves, similar analyses were repeated for the piles in all other centrifuge tests. Figure 5 compares the peak bending moments in each instrumented pile from the centrifuge tests to the corresponding bending moments estimated using *LPILE*. It can be observed that bending moments can be reasonably predicted in piles subjected to liquefaction and lateral spreading loads using the modification made to the API sand curves. The majority of the peak banding moments from centrifuge tests occurred when the wharf deck was moving in the downslope direction. The bending moments below the mudline are plotted in blue and the bending moments above the mudline (at pile head) are plotted in red, showing that the p-y models were more accurate in estimating the bending moments at pile head.

6. CONCLUSIONS

The results of five centrifuge tests on pile-supported wharves in saturated sands were used to back-calculate representative static and dynamic p-y curves for laterally loaded piles. Two types of piles were used in this study: 1) static piles loaded with various cycles of shear loads applied at the pile head prior to shaking, and 2) dynamic piles that supported the wharf deck and were subjected to deck inertia and liquefaction-induced lateral spreading during earthquake shaking. The primary conclusions of the analyses are summarized as follows:

- Back-calculated p-y curves from static piles were approximated using API sand curves. The friction angles of 33°, 37° and 45° were used for loose sand (D_R = 30% to 40%), dense sand (D_R = 70% to 85%) and rockfill, respectively. These friction angles appeared to be adequate in estimating the ultimate lateral resistance (P_{ult}) of the experimental p-y curves and no modifications were necessary. However, the stiffness values of the p-y curves that were back-calculated from the centrifuge tests were softer than those recommended by API. The back-calculated moduli of subgrade reaction were 3500 kN/m^3, 3500 kN/m^3, and 5200 kN/m^3 for loose sand, dense sand and rockfill, respectively. These values are smaller than the values recommended by API (1993) by a factor ranging from 5 to 14, which might be attributed to the limitations of centrifuge tests in simulating the in situ soil stiffness and pile driving conditions.
- The *p*-multipliers (P_m) in fully/partially liquefied zones, which were calculated based on the pore water pressure ratio (R_u) generated during dynamic loading using $P_m = 1 - R_u$, were applied to account for the softer response in liquefied zones.
- The comparison of the recorded pile bending moments and those estimated from *LPILE* demonstrates that the recommended modification on API sand curves can reasonably predict the maximum bending moments along the piles due to liquefaction-induced lateral spreading.

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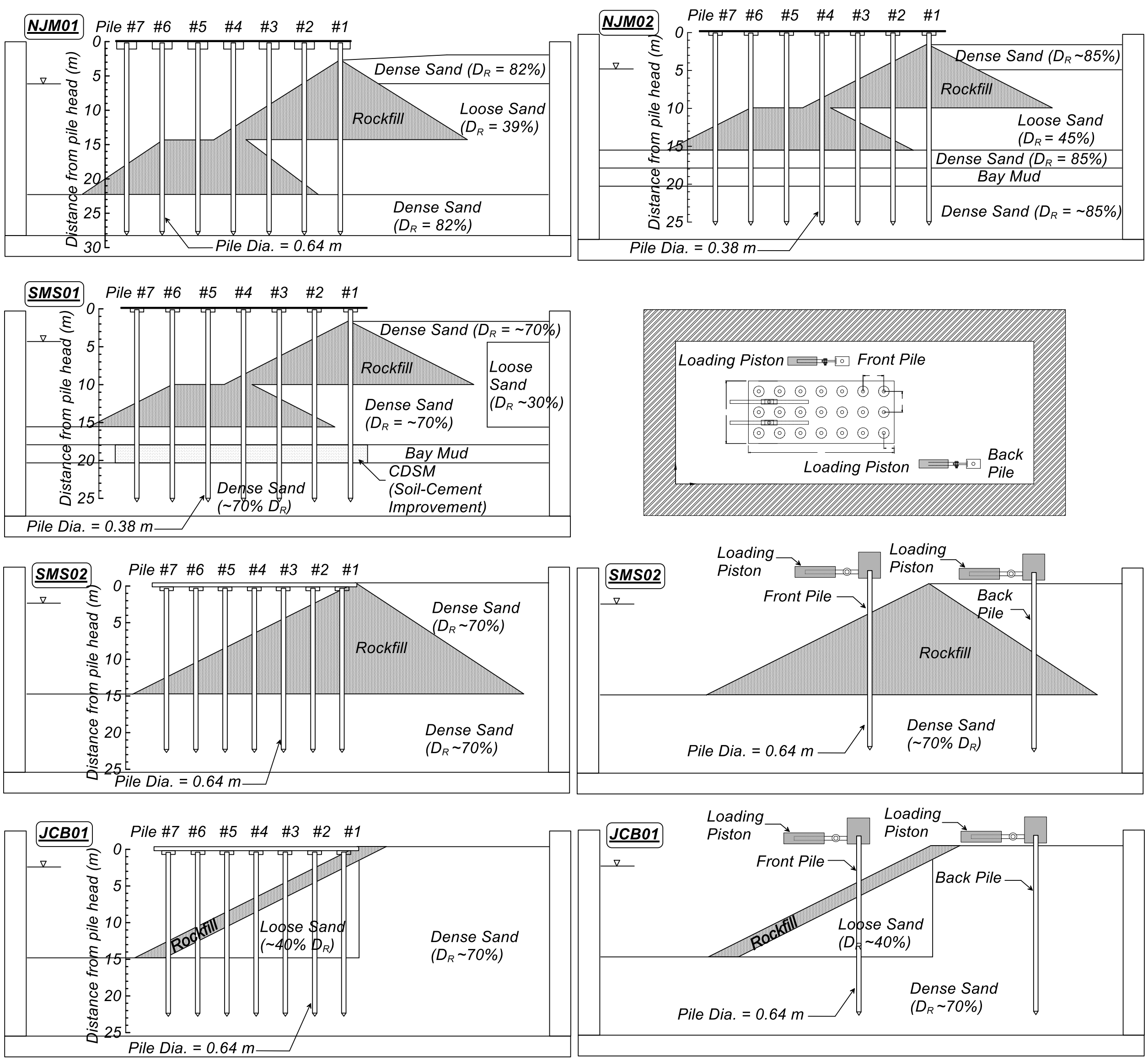

Fig. 1. Cross sections and plan view of five centrifuge tests on pile-supported wharves.

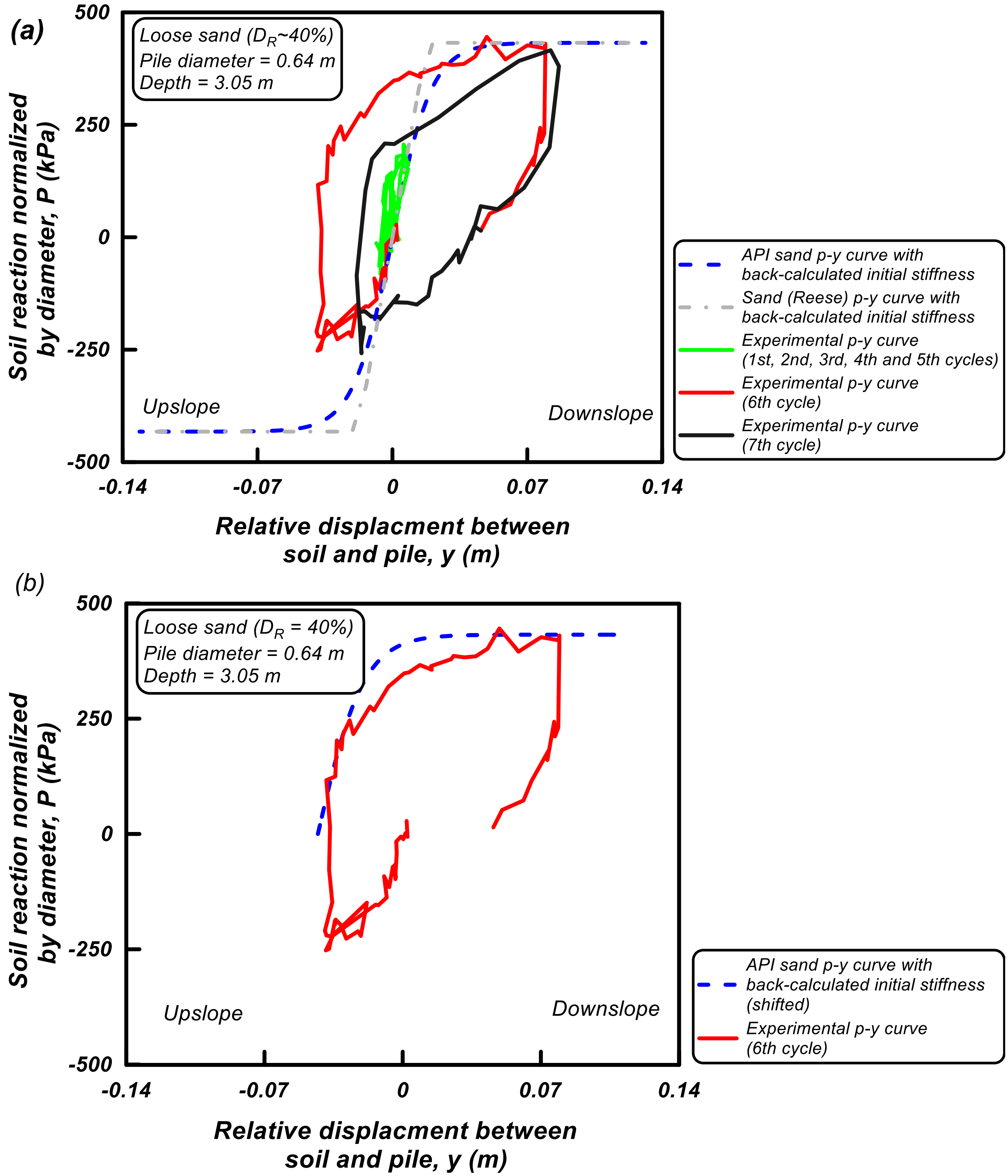

Fig. 2. Comparison of experimental p-y curves for loose sand (D_R = 40%) from the front static pile in JCB01, API sand and sand (Reese et al. 1974) using back-calculated input parameters.

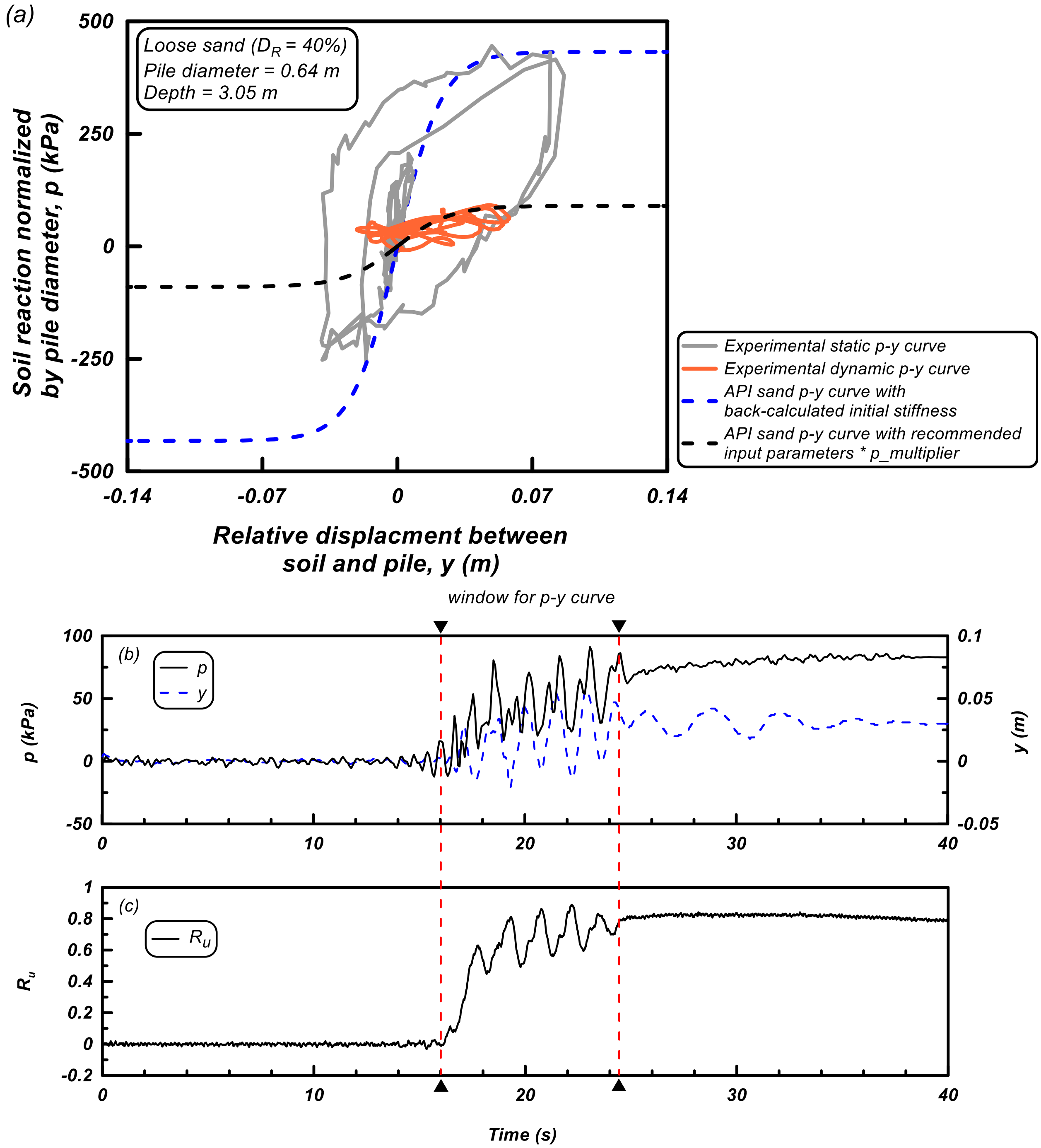

Fig. 3. (a) Comparison of static versus dynamic p-y curves in loose sand; (b) Time histories of back-calculated soil reaction and pile displacement relative to adjacent soil; (c) Excess pore water pressure ratio measured in loose sand in JCB01.

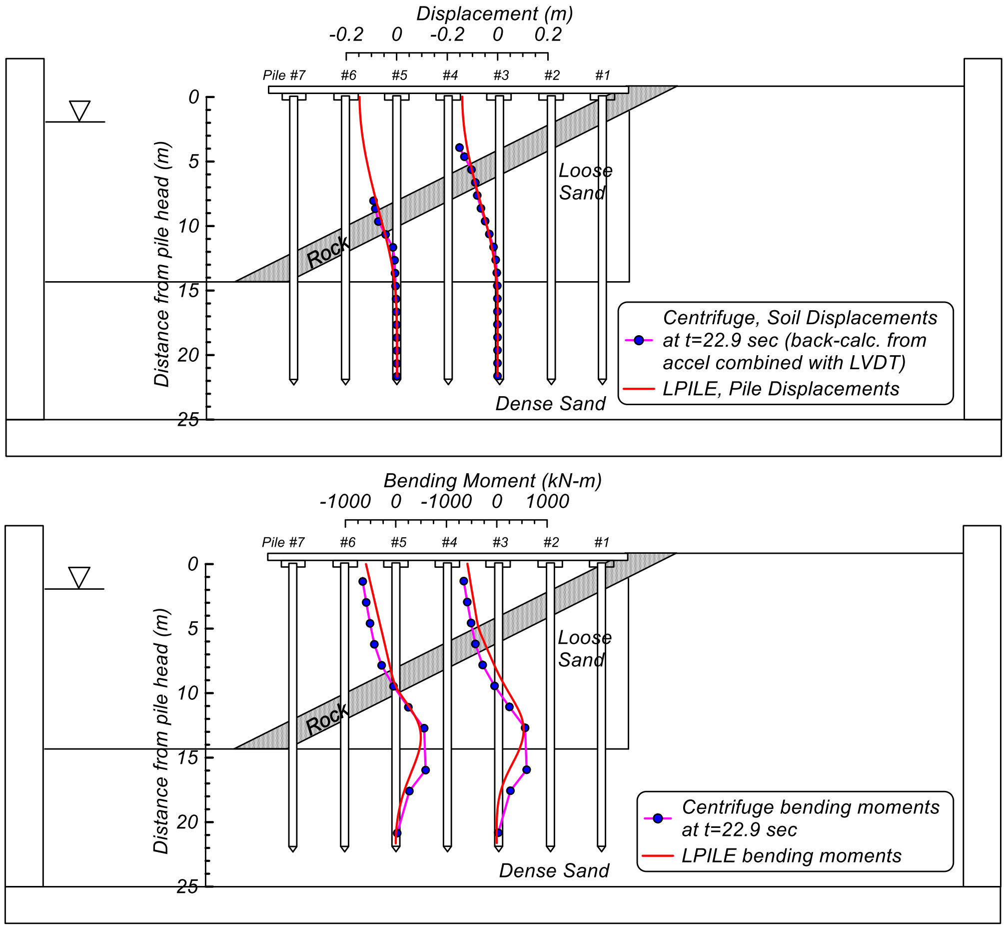

Fig. 4. (a) Back-calculated soil displacement profiles at the pile locations from centrifuge tests with estimated pile displacement profiles from *LPILE;* (b) Comparison of bending moments from centrifuge tests to the estimated moments from the *LPILE* analyses for JCB01.

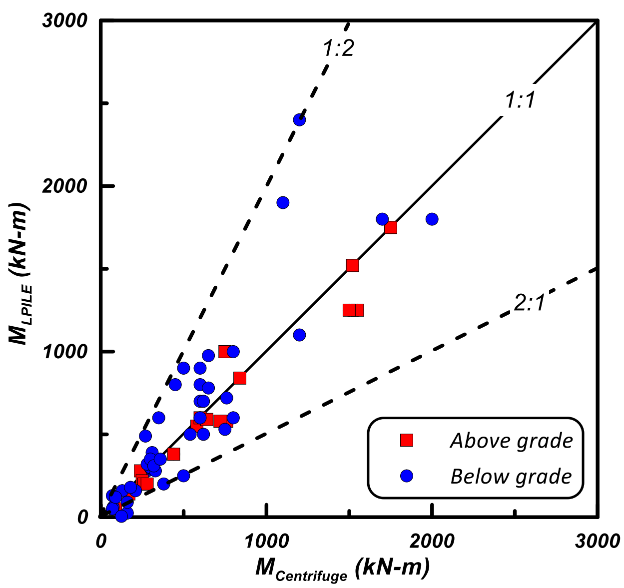

Fig. 5. Comparison of bending moments recorded from centrifuge and estimated from the *LPILE* analyses for all five centrifuge tests.