MULTI-DIRECTIONAL P-Y CURVES FOR SOFT CLAYS

Observed free field ground displacement paths during earthquakes

Soil-pile relative moments

Bidirectional p-y curve

MULTI-DIRECTIONAL CYCLIC P-Y CURVES FOR SOFT CLAYS

Juan M Mayoral¹, Juan M Pestana² and Raymond B. Seed³

¹Researcher, Geotechnical Department, Institute of Engineering, National University of Mexico, Building N° 4, P.O. Box 04510, Mexico City, Mexico, JMayoralV@iingen.unam.mx

²Associate Professor, University of California, 440 Davis Hall, Berkeley, CA 94720-1710.

³Engineer Professor, University of California, 440 Davis Hall, Berkeley, CA 94720-1710.

ABSTRACT

Currently there is a lack of information and experimental data to formulate p-y curves in multi-directional loading conditions for soft clays. Experimental evidence has shown that uni-directional p-y curves are not able to represent correctly the near field soil response to multi-directional earthquake loading. This paper presents the results of an experimental study aimed at characterizing soil-pile interface response for multi-directional pile trajectories in soft clay. Series of multidirectional p-y test were performed including circular, elliptical, parabolic, and figure-8 pile transit paths. These tests provide information regarding the force-displacement relationship dependence on the loading path, the evolution of the soil stiffness during multi-directional cyclic loading, the effect of loading history on the near field soil stiffness, and the variation of the lateral load with pile depth. The tests results showed that the multi-directional movement of the pile produces loading induced anisotropy, which can significantly affect the behavior of the near field soil. From the results gathered it is clear that the soil-pile interaction in soft clay during multi-directional loading, is complex and cannot be modeled with methods based on the traditional concept of p-y curves simply applied in two orthogonal directions.

KEYWORDS: p-y springs, lateral loading, piles, soil-structure interaction, soft clay behavior.

Contact person and address:

Juan M. Mayoral

COORDINACION DE GEOTECNIA
INSTITUTO DE INGENIERÍA
UNIVERSIDAD NACIONAL AUTONOMA DE MEXICO
MEXICO CITY, MEXICO

PHONE: +(52) 55 5623-3600 EXT: 8469

FAX: (510) 642-7476

E-MAIL: jmayoralv@iingen.unam.mx

1. Introduction

Seismic soil-pile-structure-interaction analyses very often rely on the concept of non-linear discrete spring elements to simulate the effect of the soil surrounding the pile (Bea et al., 1984; Lok et al., 2000; Mayoral et al., 2002). In particular for lateral soil-pile interaction, the so-called p-y curves are used to describe the response of non-linear springs connected to structural elements (i.e. pile), which represent the soil immediately adjacent to the pile (i.e. near field). The p-y approach was proposed by McClelland and Focht (1958) to describe the relationship between the lateral forces developed on a pile, "p", by the adjacent soil, when the pile moves a lateral displacement, "y", at a given elevation. Procedures to obtain this relationship fall into one of two categories: (a) in situ testing of full-scale single piles or pile groups (Kim and Brungraber, 1976; Brown et al., 1987; Reese et al., 1974; Cox et al., 1983) and (b) laboratory testing using scale model testing or element testing, commonly referred as pot testing (Matlock 1962a & 1970b; Lok et al., 2000, Mayoral et al., 2005). Full-scale pile testing is costly, and in many cases the results are proprietary (i.e. Bea, 1988). As a result, significant effort has been made over the last two decades to develop reliable laboratory-based techniques. In model testing (e.g. 1-g or centrifuge testing), the actual soil-pile system is modeled using appropriate similitude laws for both soil and pile to correctly simulate the actual field conditions (Riemer and Meymand, 1996; Meymand, 1998; Finn and Gohl, 1994; Wilson, 1995). In the so-called "pot" testing, it is assumed that the soil-pile system can be divided into units where the force vs. displacement distribution on a given segment of pile is uniform with depth (e.g. Matlock, 1962; Lok, 1999). Although empirical p-y curves are soil specific, generic uni-directional p-y curves have been proposed by a number of authors for both clays (e.g., Matlock et al., 1970; Reese et al., 1975) and sands (e.g., Bogard and Matlock, 1980; Reese et al., 1974; O'Neil and Muchison, 1983). Some specific uni-directional p-y curves have been incorporated into computer programs that are widely used in offshore engineering applications (e.g. Reese, 1977; Matlock et al., 1978), as well as in the design of important onshore structures such as tall bridges and buildings subjected to seismic loading (e.g. Matlock et al, 1979; Bea et al, 1988; Wang and Reese, 1993). A major advantage of using p-y curves is the ability to capture key features of soil-pile-structure interaction, such as (a) progressive gapping, (b) cyclic degradation of the soil, and (c) pile slippage with less computational effort than solutions based on continuum models. These results have led to the American Petroleum Institute recommended method for constructing p-y curves (API, 1993). Most of this work has involved either monotonic or cyclic loading along a single loading axis. Accordingly, there is a lack of information (experimental data) to formulate and calibrate p-y curves in multi-directional loading conditions. Many procedures used to evaluate the seismic response of pile foundations subjected to two-directional and three-directional components of strong shaking generally assume that the soil foundation can be substituted by a system of orthogonal

non-linear inelastic springs, with force-displacement properties given by uni-directional p-y curves. However, experimental evidence (Mayoral et al., 2002; Mayoral et al., 2005) has shown that uni-directional p-y curves are not able to represent correctly the near field soil response to multi-directional earthquake loading. In this paper, the testing protocol for determining multi-directional p-y curves described by Mayoral et al. (2005) using a bi-directional pot testing device was apply to obtain experimental data regarding the near field soil response to multi-directional relative movements between model clay and a model pile. Several series of multi-directional pot tests using simplified pile transit paths (e.g., circular, elliptical, figure-8, and parabolic) were conducted. These pile transit paths are idealizations of the deformation patterns observed in recent earthquakes (c.f., Fig. 1). Figure 1 shows some examples of normalized “free-field” ground displacement paths (also referred to as orbits) recorded in recent earthquakes. The x and y horizontal components of these paths are normalized by the Peak Ground Displacement, PGD, defined as the maximum displacement in one of the two components. These examples cover a wide range of PGDs from approximately 3 to 90 cm for earthquakes with moment magnitudes ranging from 6.5 to 7.0. Although the orbits are very complex, the largest “loop” can be approximated by one (or a combination of several) simplified displacement paths, such as: a) linear, b) elliptical/circular, c) figure-8 and d) parabolic. In here, it is assumed that the relative displacement will follow conceptually similar, but not identical, paths and thus study of these scenarios can provide insight into the overall behavior of the foundation system under multi-directional excitation. The relative displacement of foundation elements (i.e., piles) with respect to the free field will depend on the relative influence of kinematic and inertial response of the foundation and structural system.

Mayoral et al., (2005) presents full details of an enhanced pot-testing device for evaluating the multidirectional p-y response of soils, which is an extension of the original pot test proposed by Matlock (1962). This previous work was mainly focus on the test validation, pointing out the differences between multidirectional and uni-directional p-y curves, for a limited number of model pile transit paths. In the current paper, the multidirectional pot testing device was used in an exhaustive laboratory program aimed at gathering key information regarding the multidirectional soil-pile response in soft clays. The results of these tests provided data and insight regarding the force-displacement relationship dependency on the loading path, the evolution of the soil stiffness during multi-directional cyclic loading, and the effects of loading history (which includes the corresponding plastic deformations) on the near field soil stiffness. Moreover, the distribution of the lateral force with depth was measured for different loading scenarios.

The evolution of the following parameters was studied to characterize the force-displacement relationship:

1. The maximum normalized lateral force, $p/(D.L.s_u^*)$, where p is the lateral soil resistance, D and L are the pile diameter and the embedment depth respectively and s_u^* is the undrained shear strength corrected for rate effects.
2. The residual normalized lateral force, $p_f/(D.L.s_u^*)$, where p_f is the lateral force acting on the model pile when the gap is fully open and stabilized.
3. The number of cycles needed to reach the stabilized regime, N .

The data gathered not only helped to gain insight into the response of the near field, but also provided detailed information for the development and calibration of a conceptual model for near field soil. This additional work is presented on a companion paper.

2. Study of the Multi-directional p-y Response for Soft Clay

The major components of the multi-directional pot testing set up are the instrumented model pile and a testing device capable of imposing a suite of prescribed soil/pile lateral relative displacements in two orthogonal directions, as depicted in figure 2. A complete description of each of these components was presented by the authors elsewhere (Mayoral et al., 2002; Mayoral et al., 2005). Table 1 summarizes the technical specifications for the multidirectional pot testing device. However, some of the key details of the multi-directional pot testing set up are revisited in the next section for completeness.

2.1 Model Clay

The model clay is an artificial soil composed of 72% kaolinite, 24% bentonite, and 4% type C fly ash. The target water content of the mixture used in these series of tests, was 110 %. Twenty minutes for partial healing of the model clay after full remolding was allowed before testing. The undrained shear strength (measured with shear vane, and corrected for rate effects) ranged between 3.5 to 3.9 kPa, for an average peripheral velocity of 190 mm/min (+/- 5 mm/min) of the vane shear. This shearing rate matches the shearing velocity of the model pile in the direction of maximum displacement. To keep the target moisture content, a volume of water of approximately 1/3 of the soil volume contained in the multidirectional testing cylindrical bucket (Fig. 2), was added to the dry mix of kaolinite, bentonite, and type C fly ash, which was remixed by hand thoroughly, until reaching a uniform consistence. Following this procedure, the moisture content did not appear to be very sensitive to small variations in the mixing water. After each test, the water content, and undrain shear strength of the soil was characterized in four

points within the sample after testing. For a given curing time, the undrained shear strength was primarily controlled by the water content, while the small strain shear stiffness was adjusted by changing the content of fly ash (Wartman, 1996; Biscontin and Pestana, 2001; Mayoral et al., 2005). The soil container was a plastic cylindrical bucket with a diameter of 510mm, and it was filled with the model soil to a height of 350mm. These dimensions correspond to an optimal container size that minimizes the effect of the boundaries in the lateral forces measured on the pile, as it is described in detailed by Mayoral et al. (2002). The maximum model pile diameter considered was 51mm. The maximum length of embedment is 251mm measured from the soil surface to the center of the conical tip, and corresponds to a nominal embedment ratio, L/D , of five (where D is the pile diameter and L is the embedment length). The maximum off-center displacement of the pile was chosen as 203mm.

2.2 Instrumented Pile

The model pile is a 50.4 mm diameter aluminum pipe with a wall thickness of 0.3 mm, which flexural rigidity, EI , is about 2945 N.m. Table 2 summarizes the scale pile properties. Thus, the model pile is rigid enough to apply a uniform lateral pressure to the soil, to develop the p - y curves based on external transducers (i.e. load cells), but also flexible enough to allow obtaining reliable internal bending moments and shear forces from internal transducers (i.e. strain gages). This pipe was instrumented with three orthogonal pairs of strain gages (c.f., Fig. 3a). These gages were placed on the pile at three elevations, 50, 125 and 200 mm, measured from the bottom of pile cap, as shown in Fig. 3b. The gages were calibrated to measure discrete values of bending moment acting on the pile for a given loading condition. From these readings, shear force and pile deflections were back-calculated. The total shear force measured with the strain gages was cross-checked with the readings from external load cells of the total force applied to the top of the pile to minimize the uncertainties in the magnitude of the lateral soil resistance measured. The maximum length of embedment of the model pile was 251 mm measured from the top strain gauge to the center of the conical tip and corresponds to a nominal embedment ratio, L/D , of five (where D is the pile diameter and L is the embedment length).

2.3 Overview of the Test Preparation

The model clay was remolded prior testing to erase aging effects. Although the mixture did not lose a significant amount of water during this process, it was necessary to moisturize it regularly to maintain the target water content. Once the soil was remixed and homogenized completely, it was leveled and the bucket was placed on the multi-directional pot-testing device. After the sample was in place, the wire potentiometers were connected, and the air supply was turned on to balance the servo-valves and to place the pile cap and the roller table in position. The air supply activate the actuators of the servo-valves shown

in Figure 2. As a result, the model pile can be loaded and/or displaced laterally independently in two orthogonal directions. Once the system was balanced, the air supply was shut down and the model pile was pushed into the container, sliding it through the pile cap. The model pile cap holds the model pile in place and prevent rotations, as depicted in figure 2. The pile cap is a 25.4-mm-thick 304.8mm×101.6mm aluminum plate that can be adapted to clamp model piles of 51mm, 25.4mm, and 12.5mm in diameter. After the model pile was in position, the air supply was turned on again and the sample was allow to stabilize for approximately 20 minutes prior testing. The horizontal pile deflection was measured with two orthogonal wire potentiometers.

2.4 Data Acquisition and Post-Processing

A total of ten channels were monitored during each of these tests: two load cells, two wire potentiometers and three pairs of strain gages. At the same time, an extra channel was made available for the vane shear device, which was used to determine the undrained shear strength for each test. Each of these transducers generates a signal that is converted into data using the control software. A filtering post-processing was necessary to clean up the signal. A three point average filter was applied to the data with a sampling rate of 20 points per second was used throughout with excellent results. The maximum number of data points, which for a given sampling rate is a function of the duration of the test, ranged between 2000 to 5000 points. The duration of the test, which depends on the number of cycles and loading frequency, varied between 10 to 90 seconds for the tests included herein.

2.5 Internal Self Consistence Verification of the Multi-directional Pot Test

The shear forces, V , acting on the pile and the lateral pile displacements, y_p , were obtained from discrete values of bending moment, using the equations of beam theory presented below:

$$k y_p = \frac{dV}{dz} \quad (1)$$

$$V = \frac{dM}{dz} \quad (2)$$

The moment-curvature relation, from elementary small-deflection beam theory, is:

$$M = EI \frac{d^2 y_p}{dz^2} \quad (3)$$

Where: M , V , EI , y_p , k and z are the bending moment, shear force, flexural rigidity, and horizontal deflection of the pile, soil stiffness and depth respectively. This set of equations represents the strong form of the boundary value problem of a beam on an elastic foundation. To compute the shear forces and pile deflections the bending moment, M , has to be differentiated once and integrated twice respectively. The numerical procedures applied during the post-processing of the experimental data to conduct these computations are presented next.

2.6 Numerical Differentiation of the Shear Forces

Numerical differentiation is prone to be unstable and sensitive to the slenderness of the function (i.e., bending moment) to be differentiated (Cook and Young, 1985). The distribution of the bending moment in a structural element (i.e. pile) depends on both the boundary conditions and loading mechanism. To have a sense of this distribution for the conditions that prevail during the pot test, a parametric study was conducted. In this study, the scale pile was discretized and modeled with beam elements and the soil was represented by springs. The properties of these springs were varied to look at three scenarios: linear soil stiffness constant with depth, linear soil stiffness increasing with depth and nonlinear hyperbolic soil stiffness varying with depth. The computer code SSTIN-1DFE, (Desai and Kuppasamy, 1978) was used in this analysis. Details of this study can be found in Mayoral et al. (2002). These analyses showed that the bending moment distribution was slender and could be considered piecewise linear around the points where the strain gages were located. Although sophisticated numerical differentiation procedures are available in the literature, based on the above considerations, the forward difference formula presented below was considered sufficient:

$$V(z_m) = \frac{M(z + \Delta z) - M(z)}{\Delta z} \quad (4)$$

Where: V , M and z are the shear forces, bending moment and depth respectively. The point z_m corresponds to the intermediate point between strain gages. Although truncation errors cannot be estimated when this equation is used, it proved to be accurate enough in this case. Figure 4 shows some comparisons between the total force computed with this procedure and the total force measured with external transducers and corrected for friction effects on the tracks for test f81213. This test corresponds to a figure-8 pile transit path with a displacement ratio, A_x/D of 0.63 and an amplitude ratio A_x/A_y of 3. As can be seen the agreement between the lateral force measured with internal (strain gages) and external (load cells) sensors is very good.

2.7 Computation of Pile Deflections

The determination of pile deflections requires the double integral of the bending moment. The numerical technique used to perform this integration was based on the midpoint rule. The equations below summarized the procedure:

$$y_p'_{z+\Delta z/2} = \frac{M_{z+\Delta z} + M_z}{2} \Delta z \quad (5)$$

$$y_p'_{z+3\Delta z/2} = \frac{M_{z+2\Delta z} + M_{z+\Delta z}}{2} \Delta z \quad (6)$$

$$y_{z+\Delta z} = \frac{y_p'_{z+\Delta z/2} + y_p'_{z+3\Delta z/2}}{2} \Delta z \quad (7)$$

where: y_p is the pile displacement and M is the bending moment.

The slenderness of the bending moment distribution again assured realistic displacement values. Furthermore, the maximum pile deflections computed with this procedure fell within the range of those predicted during the parametric studies (Mayoral et al., 2002). Moreover, during the tests it was observed that the pile moved mostly as a rigid body. Therefore, the relative values of the pile deflections with respect to total displacements of the pile were expected to be very small. This was not surprising because the flexural rigidity of the pile was selected to be significantly larger (3 orders of magnitude) than the soil stiffness in order to ensure a uniform displacement distribution.

3. Parametric Studies

A total of thirty-seven tests were performed to identify the main factors that control the multi-directional response of the near field soil. The results for all tests are presented in Mayoral (2002). This section discusses the key aspects of selected tests. All of these tests were displacement controlled. This allowed study of the effect of the evolution of the gap on the stiffness degradation during cyclic loading. Moreover, a displacement controlled test facilitates the normalization of the displacement amplitude with respect to the pile diameter, as traditionally has been done (e.g., Matlock, 1962). The multi-directional pile trajectories utilized are circular, elliptical, figure 8 and parabolic pile transit paths. In the results included herein, the lateral forces were normalized with respect to the corresponding undrained shear strength (determined with shear vane) corrected by rate effects, s_u^* , times the pile diameter, D , times the embedment depth, L , of the pile. The horizontal displacements of the model pile in the x and y directions were normalized with respect to the pile diameter.

3.1 Effects of the Displacement Ratio

The objective of the first series of tests was to observe the effects of the displacement amplitude, A_x/D and A_y/D , and the amplitude ratio, A_x/A_y in the multi-directional p-y response of the near field. A_x and A_y are the maximum amplitudes of the pile transit path in the x and y directions respectively. From these tests it was also possible to evaluate the number of cycles required for stabilization of the p-y curve. The magnitude of the total displacement amplitudes utilized ranged between 0.25 to 1.5 times the pile diameter. The frequency of loading was varied accordingly to keep the shearing velocity along the maximum displacement of the pile between 140 to 195 mm/min. The peak strengths ranged between 3.65 to 3.85 kPa. The amplitude ratios, A_x/A_y , considered were 6 and 3. Twenty cycles of loading were applied in each test. The average water content of the model clay in these tests ranged between 104 to 116 %. Although the pile transit paths used were figure 8 and parabolic, only the results obtained for figure 8 transit paths were included herein due to space constraints. Table 3 summarizes the test information and soil properties of these test. Figure 5 shows some of the results found regarding the effect of the maximum displacement ratio, A_x/D , for a figure-8 pile transit path for three displacement ratios: $A_x/D=0.38, 0.63$ and 0.75 . As can be seen in Figure 5, the amplitude ratio, A_x/A_y was kept constant and equal to 3. As can be seen, for increasing displacement ratio, A_x/D , the lateral soil resistance tends to diminish. This is a direct effect of the evolution of the gap during cyclic loading. For displacement amplitudes, less than 0.6 the gap tends to close whereas for displacement amplitudes larger than 0.6, the gap tends to remain open.

The lateral force available at the central portion of the gap is larger if the gap tends to close than if the gap remains open. This also affects the number of cycles needed to achieve stabilization, which decreases from 9 to 5 when A_x/D increases. The magnitude of the peak lateral forces was not significantly affected by A_x/D , because the maximum soil resistance was reached at a displacement lower than that of the minimum A_x/D utilized during the test (i.e., $A_x=0.38.D$). The effect of the displacement ratio, A_y/D , for the same set of tests is presented in Figure 6 for three displacement ratios: $A_y/D=0.13, 0.20$ and 0.25 . Note that in this case, the p-y response measured is not significantly affected by the gap geometry. In fact, the size of the hysteretic loop seems to be proportional to the value of A_y/D . For increasing displacement ratio, A_y/D , the area inside the hysteresis loop increases. Similarly, the number of cycles needed to achieve stabilization is practically constant and equal to 5. Again the magnitude of the peak lateral force was not significantly affected by A_y/D .

3.2 Effects of the Amplitude Ratio

Figure 7 shows the effect of the amplitude ratio, A_x/A_y , for a figure-8 pile transit path for two amplitude ratios: 3 and 6. The displacement ratio, A_x/D was kept constant and equal to 0.63. As can be seen, the overall response is not affected by the aspect ratio. For the larger amplitude ratio (i.e., 6), that corresponds to a more elongated figure-8, the response resembles more the typical response observed in uni-directional p-y behavior. When the amplitude ratio decreases the effects of the gap geometry become more important and the observed p-y behavior deviates from the traditional uni-directional p-y response. Again, the magnitude of the peak lateral forces was not significantly affected by the amplitude ratio, A_x/A_y . Similar results were obtained for parabolic pile transit paths (Mayoral et al., 2002).

3.3 Effects of Loading History

Several series of tests were performed to study the effects of loading path and loading history on the force-displacement relationships for the near field soil (Figure 8). In the first series of tests two double-phased tests were conducted. In the first double-phase tests, 15 cycles of an elliptical displacement path followed by 10 cycles of a circular were applied to the model pile. During the first phase (i.e., elliptical displacement path), the maximum displacement amplitude, A_x/D , was equal to 0.63 and the amplitude ratio, A_x/A_y , was 0.21. During the second phase (i.e., circular displacement path), the maximum displacement amplitude A_x/D was 0.43 and the amplitude ratio, A_x/A_y , was 1. Although a slight decrease in the soil stiffness and in the number of cycles needed to achieve stabilization was observed during the second phase, the effects of the loading history were not significant for this level of relative deformations, this amplitude ratio and this pile transit path.

In the second double-phased tests, 15 cycles of a figure-8 displacement path followed by 10 cycles of an elliptical pile transit pile were applied to the model pile. As can be seen, the force-displacement relationship for the figure-8 and the elliptical pile transit path follow the same trends for the x direction and are indeed pretty close to each other for the y direction. Comparing the observed response for the elliptical pile transit path without previous loading (Figure 8a, first phase) and with previous loading (Figure 8b, second phase), it can be seen that the loading history significantly affects the final shape of the multi-directional p-y curve for these levels of displacement.

3.4 Radial Stiffness Degradation

To look at the effects of the loading history on the radial stiffness degradation due to cyclic loading, a second series of tests consisting of three-phased tests was performed. In these tests, 3 series of 15 cycles of elliptical pile transit paths were applied to the model pile. In the first series, the displacement ratio, A_x/D , was 0.38. In the second series, the displacement ratio was 0.63. Finally, in the last series, the displacement ratio was 0.75. The amplitude ratio, A_x/A_y was kept constant and equal to 3. The results are summarized in Figure 9. Note that the hysteretic response associated with the direction of maximum deformation is greatly affected by the previous loading.

The peak lateral forces observed during the second and third phases are 25 to 35 % smaller than those observed during the first phase. Also, at the central portion of the p-y curve corresponding to the third phase of the test, an area of low lateral resistance can be observed. This is associated with the gap geometry that also affects the number of cycles needed to achieve stabilization, which decreases from 4 to 1 for phase 1 and 3 respectively.

3.5 Effects of Radial Loading

Several set of series of multiphase tests were performed to look at the effects of radial loading on the degradation of the soil stiffness (Figure 10). In the first set of tests five series of 10 cycles of a figure-8 displacement path were applied to the model pile, changing the orientation of the displacement path rotating the bucket at 0, 15, 30 and 45 degrees with respect to the x direction. The second set of tests was similar, except that the displacement path used was a parabola. Similarly, in the third set of tests, 15 cycles of a figure-8 type of loading path were applied to the model pile, changing the orientation of the displacement pattern by rotating the soil container at 0, 30 and 60 degrees. Finally, in the four set of tests, 15 cycles of a parabolic displacement path were applied to the model pile, changing the orientation of the displacement pattern by rotating the soil container at 0, 30 and 60 degrees.

The objective of these series of tests was to look at the effects of the cyclic loading on the degradation of the adjacent soil and to define an area of influence around the actual pile trajectory where the properties of the soil were significantly affected by previous radial loading. The collected data were also useful to define the effects of the loading trajectory on the soil behavior. The stiffness degradation seems to be more a function of the history of loading rather than of the gap geometry. Thus, when the bucket was rotated at different angles, the progressive degradation of the soil stiffness continued until it reached stabilization. Therefore, the cyclic loading oriented at 0° affected the stiffness of the soil observed at 15° . Similarly, the stiffness of the soil at 30° was affected by the previous loading at 15° deg and so on until the stabilized regime was reached. The stabilization of the soil stiffness corresponds to a fully open gap in

which the friction force is the predominant force acting on the pile if the dragging component is neglected.

4. Variation of Lateral Force with Depth

As described before, the shear force acting over the pile was obtained at three elevations, 50, 126 and 202mm, measured from the bottom of the pile cap. Figure 11 show the corresponding curves for a particular case of a figure 8 pile transit path. The displacement ratio for the figure-8 displacement path was 0.63. The amplitude ratio was 3 in both cases. It can be seen that the upper portion of the pile (1.5 to 3 pile diameters) carries much of the load (approximately 60%), which rapidly decreases with depth. Although the trends of the hysteretic loops at several elevations are similar, it can be observed that the response becomes more plastic when the depth increases. This is expected, because the gap tends to close at depth due to the increase of the confinement pressure on the adjacent soil. Similar results were found for other pile transit paths (Mayoral, 2002), but were not included herein due to space limitations.

5. Variation of Contact Force at the Soil-Pile Interface

To understand better the differences between the uni-directional and multi-directional p-y response, and the dependency of the soil stiffness on the pile transit path, the distribution of the contact force at the soil-pile interface was studied. Figure 12 shows this distribution for the first cycle of a circular, an elliptical and a figure-8 pile transit path, respectively. As can be seen, there is a direct correlation between the inclination of the contact force with respect to the pile trajectory and the shape of the hysteresis loop. Thus, when the contact force is aligned with the pile trajectory the force-displacement relationship resembles more uni-directional loading. This condition occurs at the central portion of the figure-8 pile transit path (Figure 12c). For the extreme case of a circular pile trajectory, the obliquity between the contact force and the displacement path remains constant for subsequent cycles of loading after the maximum displacement amplitude is reached.

The contact force distribution seems to be a direct function of how the gap evolves in each case. For instance, for a figure-8 path, the gap has the classical oval structure observed in uni-directional loading, but now the width of this gap is not only a function of the pile diameter but also of the displacement amplitude in the transverse component of motion. The interaction between passive pressure, friction and dragging force is more complex in comparison with that of uni-directional behavior. Every time that the pile is opening the gap, a passive, friction and dragging force are acting, until the pile changes direction towards the inner part of the gap. Then the passive force vanishes and the friction component tends to decrease until the dragging force is the only one present. After a while the pile makes contact with the soil

again and the sequence is repeated. In general, the geometry of the gap formation, which is function of the path trajectory, will dictate how the passive, friction and dragging forces will interact.

6. Multi-directional p-y Curves

To construct multi-directional p-y curves, (i.e., p_{sx} -x, and p_{sy} -y, where p_s is the soil reaction at a given elevation) the relative displacements x and y had to be determined. The relative displacements were computed by subtracting the pile displacements, x_p and y_p , (obtained by double integrating the discrete values of bending moment measured with the strain gages) from the free field displacements, x_{ff} and y_{ff} . The free field displacements were measured with an orthogonal set of wire potentiometers (Mayoral 2002; Mayoral et al., 2005). The lateral soil reactions for a certain depth, p_{sx} and p_{sy} , were computed subtracting the load carried by the pile at a given elevation, p_x and p_y , from the total force applied. The p-y curves were obtained at two elevations: 88 and 164 mm measured from the bottom of the pile cap. Typical multi-directional p-y curves are presented in Figure 13 for a figure-8 and an elliptical displacement path. The results for other pile transit paths are compiled in Mayoral 2002.

For completeness, the experimental multi-directional p-y curves were compared with those computed following the criteria proposed by the American Petroleum Institute (API). This comparison is included in Figure 13. As can be seen, the API recommendations can be used to predict with some degree of accuracy the peak force during the first branch, but fails to capture the overall trend of the experimental p-y curves.

6.1 New insights into the behavior of p-y response

The new testing methodology was used to perform a systematic study of the force-displacement relationship for multi-directional loading of model clay-model pile interfaces. The displacement paths included were parabolic, elliptical, figure 8 and circular path in plan view. The characteristics of these paths including displacement ratio, amplitude ratio, shape and orientation were varied in order to model a larger spectrum of loading scenarios. The results obtained for multi-directional pile transit paths show that the maximum normalized lateral force and the initial tangent stiffness are independent of the loading path. However, the shape of the p-y curve is affected by the loading path, and is a function of the evolution of the gap and soil degradation during the cyclic loading. The number of cycles of loading required to achieve full stabilization of degradation (and gapping) for multi-directional pot test was path-dependent as well. The data gathered showed that the multidirectional movement of the pile with respect to the near field has the following effects on the clay-pile interface response:

- 1) Loading induced anisotropy, which can significantly affect the behavior of the near field soil.

2) Generation of preferential deformation directions due to previous loading history that is controlled by the associated gap and softened soil zone.

The residual force after stabilization seems to be independent of the loading path. The reduction of the number of cycles needed to reach stabilization appears to be controlled by the accumulation of plastic deformations with progressive multi-directional loading. The response of the near field soil is often represented with uni-directional p-y curves. However, two-directional lateral movements of the pile can cause radial degradation of the soil stiffness and create radial gapping, which is not captured by the one dimensional p-y representation. Hence, although one may obtain results by applying forces and rotations (total six) from one-dimensional uncoupled solutions, it would not represent full three-dimensional solutions. Currently, there is a lack of information regarding multi-directional p-y response. The correct characterization of the radial stiffness degradation and gapping is crucial because this effect reduces the confinement of the pile, leading to an increase in the bending moment on the structural foundation member as well as a typically larger demand in the superstructure. The experimental evidence showed that the multi-directional movement of the pile process loading induced anisotropy, which can significantly affect the behavior of the near field soil. History of loading also has an important effect on the behavior of the near field soil. Plastic soil deformation associated with previous loading affects the soil response for subsequent pile displacements. However, no change was noticed in the residual force reached after stabilization. The reduction of the number of cycles needed to reach stabilization appears to be controlled by the accumulation of plastic deformations with progressive multi-directional loading.

The contact force can be decomposed into passive, friction and dragging components. The dragging force is function of the boundary conditions at the pile tip, and for practical purposes is very small and can be neglected. For multi-directional loading the experimental evidence proved that the interaction between these forces is complex and cannot be model with methods based on the traditional concept of p-y curves simply applied in two orthogonal directions.

6.2 Applicability of the multidirectional p-y curves

In practice, the structural seismic response can be obtained using uncoupled or coupled analyses. In the uncoupled analysis, the dynamic interaction between the structure and the soil-foundation is neglected. Ground displacement time histories are obtained without the presence of the structure. Once these displacements are known, they are applied as input at the base of the structure through dynamic impedances (i.e. springs and dashpots) that represent the near field soil domain. In contrast, the coupled approach explicitly considers both the dynamic and kinematic interaction between the soil foundation and the structure during the solution process of the boundary value problem. The coupled approach provides a

more realistic solution to the seismic soil-pile-structure interaction problem for cases in which a large degree of nonlinearity is expected in the near field, but requires more analytically sophisticated methods. In order to capture the highly non-linear behavior of the near field soil during lateral loading, the near field soil can be represented by an orthogonal (horizontal) set of non-linear springs elements, also referred to as p-y spring elements. The stiffness of these nonlinear springs is obtained by multi-directional p-y curves. The use of p-y curves allows modeling of the gapping and slippage between the pile and the surrounding soil. These discrete elements are generally used in time domain solution algorithms, which allow the updating of the soil properties (i.e., stiffness and damping) according to the level of relative displacements between the soil and the pile. Many computer programs widely used in the design of deep foundations follow this procedure for uni-directional loading. The multi-directional “pot” testing device allows the study of soil-pile units to better characterize the multi-directional response of the near field during cyclic loading. The test results obtained from these tests fill an important gap in experimental information regarding multi-directional p-y curves. The multidirectional pot test is aimed at obtaining soil-specific p-y curves for a pile segment within a given soil layer. Thus, once the soil-pile segment interaction was characterized at this layer, if soil conditions changed in the next one, this can be taken into account on the numerical model using the appropriate p-y curve. A simplified discrete model for the near field soil formulated in the time domain can be proposed to simulate the response observed during multi-directional loading of soil-pile segments. In this type of formulation, pairs of nonlinear springs and linear dashpots are employed to represent the interface soil. Numerically, the stiffness of each nonlinear spring can be described using several elasto-plastic springs that can be arranged in series or in parallel. The configuration in parallel decreases the number the degrees of freedom and is more stable during the numerical computations (Lok et al., 2000). The stiffness of the nonlinear springs can also be approximated by a mathematical function (Kagawa, 1983), often of hyperbolic nature, which relates the lateral strains in the near-field soil with the corresponding stresses. However, these types of models cannot capture gapping or slippage and a more flexible approach, such as the discrete type of models, are better suited to large displacement cases. Nonlinearities associated with gapping and slippage of the pile during two-directional loading are well captured overall by this type of formulations.

A discrete model must be able to describe the evolution of the stiffness in the transition regime, accounting for the progressive formation of the gap around the pile. The description of this transitional state can be achieved with the introduction of a function (damage-like) that can be adjusted to better model the accumulation of plastic deformations (gapping) during cyclic loading. This allows a better characterization of the energy dissipated during transient dynamic loading. Once the gap is fully open and stabilized, the model is able to describe the steady state response, including the passive component and

the flat central portion of generic p-y curves reported in the literature for cyclic loading. In general, the model should be able to capture the trends, initial stiffness, peak and residual forces very closely. Although radiation damping is not incorporated explicitly in the formulation, the nonlinear spring can be used in series with a dashpot to model radiation damping (i.e., geometric damping) (e.g., Lok, 1999; Lok and Pestana, 2000). In any case, the spring stiffness is frequency independent. Rate effects can be taken into account by properly correcting the shear strength, during the construction of the p-y curve, according to the relative shearing velocity in the interface soil-pile (Lok, 1999). The hysteretic damping is introduced directly by the non-linear spring, and the radiation damping is provided by the added viscous-dampers. The computation of the pile response over time is carried out with an implicit finite difference marching in time algorithm. A simplified approach to model clay-pile interface behavior during multidirectional loading will be presented in a companion paper.

7. Conclusions

There is currently a lack of experimental data to formulate and calibrate p-y curves in multi-directional loading conditions. This paper presents a parametric study of the force-displacement relationship for multi-directional loading of the near field that was performed using the new testing methodology and the corresponding results are presented in this chapter. The displacement paths employed included parabolic, elliptical, figure 8 and circular pile transit paths. The characteristics of these paths, e.g., displacement ratio, amplitude ratio, shape and orientation were varied in order to model a larger spectrum of loading scenarios. The resulting experimental data enhanced the understanding of the multi-directional behavior of the near field soil, and laid the ground for the formulation of a conceptual model for the near field.

The experimental evidence showed that the multi-directional movement of the pile produces loading induced anisotropy, which can significantly affect the behavior of the near field soil. History of loading also has an important effect on the behavior of the near field soil. Plastic soil deformation associated with previous loading affects the soil response for subsequent pile displacements. However, no change was noticed in the residual force reached after stabilization. The reduction of the number of cycles needed to reach stabilization appears to be controlled by the accumulation of plastic deformations with progressive multi-directional loading.

The contact force can be decomposed into passive, friction and dragging components. The dragging force is function of the boundary conditions at the pile tip, and for practical purposes is very small and can be neglected. For multi-directional loading the experimental evidence proved that the interaction between these forces is complex and cannot be modeled with methods based on the traditional concept of p-y curves simply applied in two orthogonal directions.

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Figure 1. Examples of “free-field” ground displacement paths observed during recent earthquakes.

Figure 2. Multidirectional Pot Testing Device

a)

b)

Figure 3. Instrumented Model Pile: a) Photograph and b) Schematic Representation

Figure 4. Cross-Checking of Lateral Force Measured with External and Internal Transducers

Figure 5. Effect of the Displacement Ratio, A_x/D , on the Multi-directional p-y Response for $A_x/A_y=3$

Figure 6. Effect of the Displacement Ratio, A_y/D , on the Multi-directional p-y Response for $A_x/A_y=3$

Figure 7. Effect of the Amplitude Ratio, A_x/A_y , on the Multi-directional p - y Response

a)

b)

Figure 8. Effect of Loading History on the Multi-directional p - y Response for a Multiphase Test with:
a) an Elliptical-Circular, b) a Figure-8-Elliptical Pile Transit Path

Figure 9. Radial Stiffness Degradation for a Multiphase Test with an Elliptical Pile Transit Path

Figure 10. Effect of Radial Loading on the Multi-directional p - y Response for a Multiphase Test with a) a Figure-8 Pile Transit Path, b) a Parabolic Pile Transit Path.

Figure 11. Variation of Lateral Force with Depth

a)

b)

c)

Figure 12. Spatial Distribution of Contact Force for: a) a Circular Pile Transit Path, b) for a Circular Pile Transit Path and c) a Figure 8 Pile Transit Path

Figure 13. Multi-directional p - y curves for: a) Test f 81213 y b) Test frcd75

Table 1. Technical specifications for the multidirectional pot testing device

Element	Characteristics	Nominal Maximum Values
Overall System	Vertical Static Force	15 kN
	Horizontal Force, in both directions	2.1 kN
	Horizontal Stroke, in both directions	0.254 m
Container	Diameter	0.51 m
	Height	0.43 m
Roller Blocks	Vertical Load Capacity: Static, (Dynamic)	15.2 (22.4) kN
	Static Moment Capacity (all directions)	0.21 kN-m
Load Cells	Axial Capacity	2.2 kN
Servo-valves	Range of Operation	620-685 kPa
DC Wire Potentiometers	Travel Length	+/- 380 mm
	Capacity at 685 kPa	5030 kN
Pneumatic Pistons	Stroke	180mm
	Bore size	100mm

Table 2. Summary of the Scale Pile Properties

Material	Young Modulus, E, (kPa)	Diameter, D (mm)	Wall Thickness, t (mm)	Flexural Rigidity, EI (N.m)
Aluminum	140	51	0.3	2945

Table 3. Summary of the Tests Information and Soil Properties

Test #	Path	A_x/D	A_x/A_y	$F_x(\text{Hz})$	F_y (Hz)	$\omega(\%)$	s_u^* (kPa)
f81375	Figure 8	0.375	3:1	0.08	0.16	112	3.78
f81312	Figure 8	0.63	3:1	0.05	0.1	108	3.78
f81315	Figure 8	0.75	3:1	0.04	0.08	112	3.82
f81213	Figure 8	0.63	3:1	0.05	0.10	110	3.84
f81612	Figure 8	0.63	6:1	0.05	0.10	113	3.88
fcc212	Elliptical	0.63	3:1	0.05	0.10	105	3.80
fcc122	Circular	0.43	1:1	0.05	0.05	108	3.28
f8c1213	Figure 8	0.63	3:1	0.05	0.10	110	3.84
f8c1214	Elliptical	0.50	5:2	0.08	0.16	110	3.84
fcrd75	Elliptical	0.375	3:1	0.08	0.16	114	3.80
fcrd12	Elliptical	0.63	3:1	0.06	0.12	114	3.80
fcrd15	Elliptical	0.75	3:1	0.04	0.08	114	3.80
f812XXB	Figure 8	0.75	3:1	0.08	0.16	112	3.84
fc9012	Circular	0.60	1:1	0.05	0.05	116	3.64
fc0012	Elliptical	0.60	3:1	0.05	0.10	112	3.82
fcrd75	Elliptical	0.375	3:1	0.08	0.16	114	3.74