LATERAL LOAD TESTS ON DRILLED PIERS IN SAN DIEGO AREA RESIDUAL AND FORMATIONAL SOILS

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This paper presents the results of lateral load tests on 0.61-m to 1.27-m (24 to 50-inch) diameter drilled and cast-in-place piers in San Diego area residual/formational soils. The test piers were installed and tested in the following soils: Decomposed Granitic Rock (Ktd), Stadium Conglomerate (Tst), and Friars Formation (Tf). Lateral loads of up to 845 KN (190 kip) were applied to the piers in residual granitic soils, and loads up to 1780 KN (400 kip) were applied to the piers in Stadium Conglomerate and Friars Formation. The tests were conducted by jacking against adjacent piers and monitoring the lateral load and lateral deflection at the ground line. Test results indicate that ultimate lateral pile capacities of up to 1800 KN (400 kip) can be obtained for 0.61 m (24-inch) drilled piers with penetration of 3 to 6 m (10 to 20 ft) in the residual/formational soils tested. Lateral load analyses using p-y curves for very dense sand significantly overestimate the observed lateral deflections. For lateral load analyses using dense sand p-y curves, p-multipliers that increase the lateral soil stiffness (p-y curves) by 2 to 8 times are required to match the load-deflection behavior measured in the test piles. When applied to the dense sand p-y curves, the p-multipliers provide a more accurate prediction of load-deflection behavior in the residual/formational soils, and can result in shorter required pier penetration and significant savings in foundation cost.

INTRODUCTION

Drilled and cast-in-place piers are extensively used to support bridges, highway structures, transmission towers, overhead rail alignments, and buildings. In active seismic areas, such as California, lateral loads govern the design of piers in many cases. Pier design for lateral loads can be based on ultimate load analysis and a factor of safety or on an allowable deflection. An accurate assessment of pier top load-deflection behavior is necessary to perform structural analysis of the superstructure under seismic conditions. A number of microcomputer-based analytical methods are available to predict load-deflection behavior of piers in sand and clay (Reese, 1977, Bhushan, et el. 1980, 81). Full-scale lateral load test data for drilled piers constructed in stiff clays and medium dense to dense sands have been presented in the literature (Bhushan, 1979, 1981).

However, no full-scale lateral load test data for highly overconsolidated formational or residual soils are available in the literature. Drilled piers are commonly installed in these types of materials, and most engineers use dense sand p-y curves to model the pile response to lateral loads. The sandy residual/formational soils tested show significantly stiffer load-deformation behavior than predicted using p-y curves for cohesionless dense sand. The use of dense sand parameters for design of drilled piers in the formational soils was expected to over-predict lateral deflections and required embedment. A load test program was proposed because it was anticipated that design parameters based on full-scale load tests could result in more realistic prediction of lateral stiffness, and a significant reduction in required pier lengths, construction time, and cost.

This paper presents the results of full-scale load tests on six drilled piers in San Diego area residual and formational soils. The tests in Decomposed Granitic Rock were performed on 1.03 and 1.27 m (40 and 50 inch) diameter piers with a penetration of 3 m (10 feet). Due to the limitations of the testing apparatus, the piers could not be loaded to more than 890 KN (200 kip) lateral load. The tests in the Decomposed Granitic soils were performed as a part of a geotechnical investigation for an alignment of 500kV-transmission line towers. The tests in Stadium Conglomerate and Friars formation were performed on 0.6 to 0.8 m (24 to 32 inch) diameter drilled piers with a penetration of 6 m (20 feet). These piers were loaded to maximum lateral loads of up to 1800 KN (400 kip). The tests in Friars formation and Stadium Conglomerate were performed in December 1999 for the Metropolitan Transit Development Board (MTDB) as part of the Mission Valley East Light Rail Trolley project. Lateral loads for the piers in Granitic soils were applied by jacking the piers towards each other. For piers in Stadium Conglomerate and Friars formation, the piers were jacked apart. Schematic test setups for the tests in DG and formational soils are shown in Figures 1 and 2, respectively.

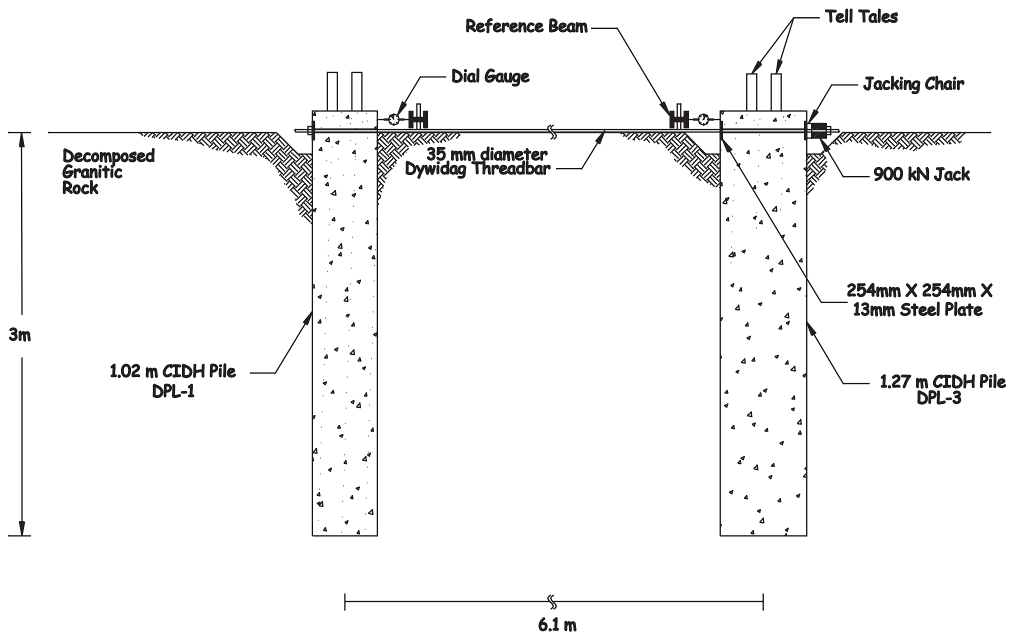

FIGURE 1 SCHEMATIC LATERAL LOAD TEST SETUP, DECOMPOSED GRANITE

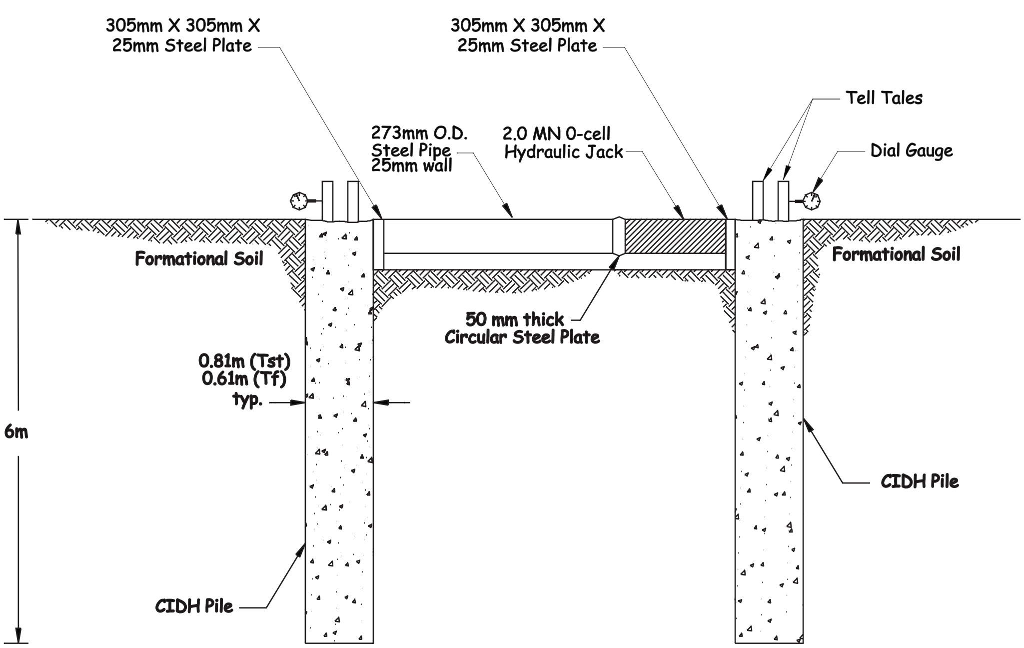

FIGURE 2 SCHEMATIC LATERAL LOAD TEST SETUP, FORMATIONAL SOILS

Measurements of groundline deflection were made for all piers, and inclinometers were used to measure deflection with depth for the piers in formational soils. Comparisons of predicted and measured deflections indicate that existing finite difference computer programs, such as LPILE (ENSOFT, 1997) and PILED/G (GEOSOFT, 1988), significantly over-predict the deflections if the highest dense sand parameters are used to develop p-y curves for the analyses.

Finite difference computer programs for lateral pile analysis typically include the provision to use "p-multipliers," or a multiplication factor that is applied to the load ("p") component of the internally generated p-y curves. Without use of p-multipliers, unrealistically high friction angles of 55 to 65 degrees must be used to match the observed behavior of the test piers using LPILE. To avoid use of unrealistic soil parameters, a friction angle of 42 degrees and dense sand k-value of 61 MN/m^3 (225 pci) as recommended in the LPILE manual were selected for analysis, and p-multipliers of 3.5 to 8 were required to match the measured behavior.

The user's manual for PILED/G recommends that for formational soil (a.k.a. "soft rock" or highly overconsolidated soil), 100% relative density and a P-multiplier of 1.25 to 2.0 should be used to predict lateral load behavior. The test results indicate that for PILED/G, relative densities of 100% and a p-multiplier of 2.0 to 5.0 must be used to match the measured deflection of the test piers.

SOIL CONDITIONS

Decomposed Granitics (Ktd)

Decomposed granitic rock is a soil generated by in-place weathering of granitic rocks of the Southern California Batholith. Unweathered and partially weathered rocks are overlain by a variable thickness of decomposed (or completely weathered) granite. The decomposed granite at the test site was a soil consisting of dense to very dense silty sand. The fines content (percent passing a No. 200 sieve) ranged between 25 and 30 percent. The measured standard penetration blow counts ranged from 30 to in excess of 50 blows per 0.3 meters (1 foot).

Stadium Conglomerate (Tst)

This formation is a tertiary-age cobble conglomerate composed (on average) of roughly 30% cobbles, 30% gravel, 25% sand, and 15% fines. Occasional boulders up to 600 mm (2 ft) are present locally. Cobbles and boulders are generally sub-rounded to rounded. Fines (soils passing the No. 200 sieve) range from silty to clayey. The matrix generally consists of a weakly cemented silty to clayey sand. Local zones of moderate cementation are occasionally encountered. Due to the high gravel, cobble, and boulder content, it is not possible to obtain meaningful Standard Penetration Test (SPT) blowcount data in this formation.

Friars Formation (Tf)

Stratigraphically, the tertiary-age Friars Formation underlies the Stadium Conglomerate in the San Diego area. The Friars formation is primarily very dense friable sandstone with occasional hard siltstone and claystone interbeds. The sandstone consists of lightly cemented silty and clayey sands with typical fines contents (percentage passing a No. 200 sieve) of 20 to 40 percent. Occasional zones of strong cementation may be encountered locally. Claystone/siltstone layers are very hard, with undrained shear strengths generally in excess of 285 KPa (6 ksf). SPT blowcounts generally range from 50 to in excess of 100 blows per 0.3 meters (1 foot). The test piers were installed in the sandstone portion of the formation.

PIER INSTALLATION

The drilled piers for the decomposed granitic soils were drilled using a Highway 29 Auger rig. The site is located close to SR 94 about 3 km west of Barrett Junction. Two piers were constructed to a depth of 3 m (10 feet). Piers DPL-1 and DPL-3 had average diameter of 1.02 m and 1.27 m (40 and 50 inch), respectively. The average strength of the concrete at the time of testing was about 24,000 KPa (3,500 psi). The modulus of elasticity of the concrete was taken as 2.32×10^7 KPa (3.37×10^6 psi).

The drilled piers at the Stadium Conglomerate site were installed with a Watson-300 drill rig equipped with a 0.61-m diameter flight auger. The site is located near the northeast corner of the intersection of Baltimore Drive and

Interstate 8, in La Mesa California. Due to the presence of cobbles and boulders, the actual variation in diameter along the lengths of the exhumed piers was measured, and ranged between 0.71 and 0.86 m (28 and 34 inches) with an average diameter of 0.81 m (32 inches). The average diameter was used in lateral load analysis. Pier TS-2 was installed in the dry. Pier TS-1 was installed with polymer slurry in the hole, and concrete was placed from the bottom with a tremie while displacing the slurry. This process was used to simulate the construction of some of the production shafts, which were to be drilled into the formation underlying a layer of alluvium below the water table. For these piers, slurry was used to stabilize the hole during construction. Due to schedule constraints, the test piers were tested four or five days after concrete was poured. The minimum required concrete strength of 24,000 KPa (3,500 psi) was achieved after three days. The modulus of the concrete was determined by performing stress-strain compression tests. Due to the use of high early strength cement, although the specified compressive strength was achieved, the elastic modulus at the time of the test was significantly lower than the modulus for normal concrete with the same compressive strength. The modulus used in the analysis was 1.27×10^7 KPa (1.84×10^6 psi).

The drilled piers at the Friars Formation site were installed with a Watson-300 drill rig equipped with a 0.61-m (24-inch) diameter flight auger. The site is located in Parking Lot X on the campus of San Diego State University. The actual diameter of the holes was close to the auger size with an average diameter of 0.61 m (24 inch). Pier TS-4 was installed in the dry. Pier TS-3 was installed with polymer slurry in the hole, and concrete was placed from the bottom with a tremie while displacing the slurry. The minimum compressive strength of the concrete of 24,000 KPa (3,500 psi) was achieved. Based on stress-strain data on the concrete cylinder, a modulus of 9.85×10^6 KPa (1.43×10^6 psi) was used in the analyses.

LOAD TEST PROCEDURE

The piers in the Decomposed Granitics were jacked towards each other by a 890-KN (200 kip) capacity calibrated jack. Lateral deflection was monitored on each pier by an independently supported dial gage located at height of about 150 mm (6 in.) above the loading rod. Loads were applied generally in increments of 89 to 178 KN (20 to 40 kip) to a maximum load of 800 KN (200 kip). One unload reload cycle was performed at about 535 KN (120 kip) load. Each load was maintained until the rate of deflection was less than 0.025 mm (0.001 in.) per minute. The schematic load test set up for the DG site is shown in Figure 1.

The piers for the Stadium Conglomerate and Friars Formation sites were jacked apart by a 1780-KN (400-kip) capacity calibrated Osterberg-Cell jack. Lateral deflection was monitored on each pier by an independently supported dial gage located at height of about 1 meter above the top of the pier. To obtain pier-top deflection, a correction was applied to account for pier head rotation. Loads were applied generally in increments of 135 KN (30 kip) to a maximum load of 1800 KN (405 kip). No unload reload cycle was performed. Each load was maintained for 10 minutes for loads up to 670 KN (150kip) and 20 minutes for loads greater than 670 KN (150 kip). A schematic load test set up for the Friars Formation and Stadium Conglomerate sites is shown in Figure 2.

LOAD TEST RESULTS

The results of the lateral load tests along with the various p-y analyses are summarized in Figures 3 through 5, which show measured pier-top deflections versus applied lateral load. The pier installation data, deflections at 445 KN, 890 KN, 1335 KN and 1780 KN (100 kip, 200 kip, 300 kip, and 400 kip) lateral load, and lateral loads at 6.4 mm, 25 mm and 50 mm (0.25 in.,1 in., and 2 in.) deflection are summarized in Table 1.

ANALYSIS OF TEST RESULTS

A number of methods have been proposed (Reese, 1977, Bhushan et el. 1980, Bhushan et el., 1981) for predicting the lateral load behavior of drilled piers. The most complete analysis, including variation of pier deflections, slopes, bending moments, and shear along the length of the pier can be made by means of a computerized finite difference solution. A number of computer programs are available, such as LPILE (ENSOFT,1997) and PILED/G (GEOSOFT,1988). In these programs, the soil lateral resistance is modeled by non-linear load deflection (p-y) curves, and the flexural stiffness of the piers is used to obtain compatibility between the pier and the soil deformations. In addition, various loading and boundary conditions can be incorporated in the analysis. We selected LPILE and PILED/G for analyzing the load test results. These programs internally calculate p-y curves based on soil input data using various formulations (Reese, 1974, Bhushan, et el. 1981). The p-y formulations were derived from limited load test data, and are generally based on user-input soil type (sand or clay), strength parameters, and correlations for the load-deformation characteristics of the soil.

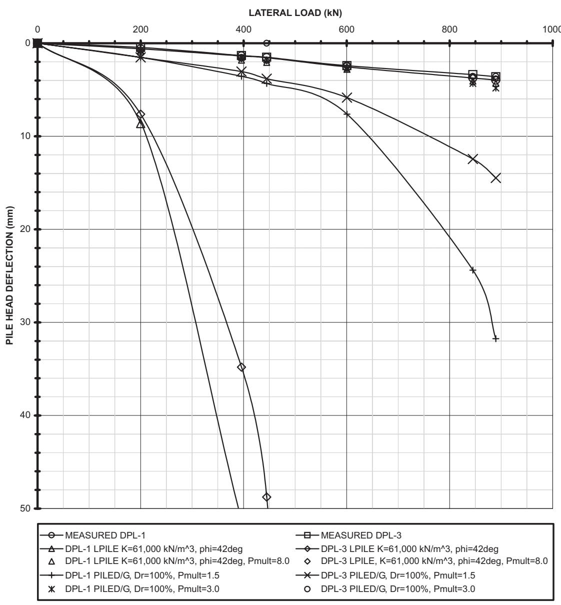

FIGURE 3 : LATERAL LOAD TEST RESULTS AND P-Y MODELS 1.02 AND 1.27 METER DIAMETER CIDH PILES IN DECOMPOSED GRANITE

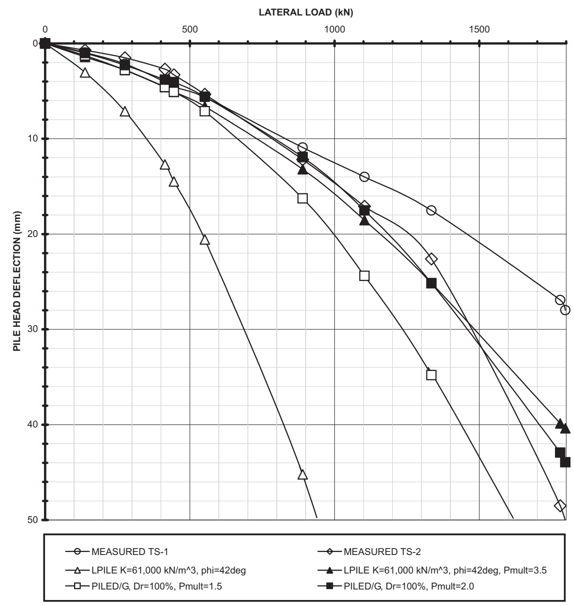

FIGURE 4 : LATERAL LOAD TEST RESULTS AND P-Y MODELS 0.81 METER DIAMETER CIDH PILES IN STADIUM CONGLOMERATE

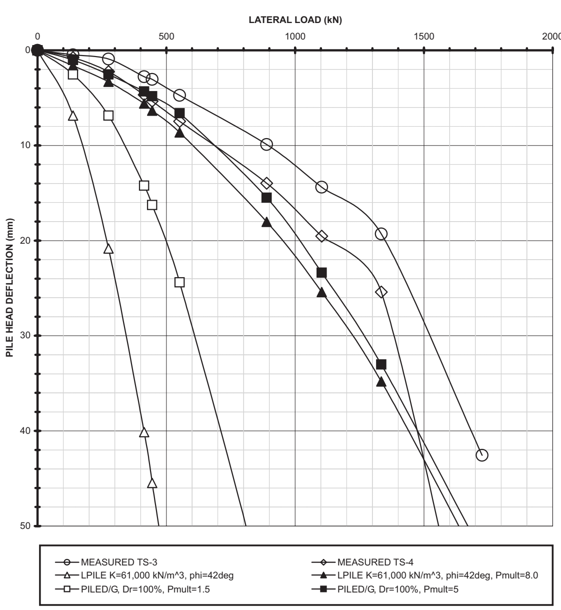

FIGURE 5 : LATERAL LOAD TEST RESULTS AND P-Y MODELS 0.61 METER DIAMETER CIDH PILES IN FRIARS FORMATION

Table 1 SUMMARY OF PIER INSTALLATION AND TEST RESULTS

Pier No.	Diam., m	Depth, m	Deflection in mm at load, KN				Load in KN at deflection, mm		
			445	**890**	**1335**	**1780**	**6.4**	**25**	**50**
Decomposed Granite									
DPL 1	1.02	3.0	1.6	3.6(1)	--	--	1245(1)	--	--
DPL 3	1.27	3.0	1.5	4.0(1)	--	--	1335(1)	--	--
Stadium Conglomerate									
TS-1(2)	0.81	6.0	4.5	11.0	17.6	27.0	600	1685	--
TS-2	0.81	6.0	3.3	12.2	22.7	51.8	600	1400	1770
Friars Formation									
TS-3(2)	0.61	6.0	3.0	10.0	19.2	42.6(3)	680	1420	--
TS-4	0.61	6.0	5.3	14.0	25.5	70.5(3)	500	1330	1550

(1) Extrapolated
(2) Test piers TS-1 and TS-3 were drilled with polymer slurry.
(3) At a load of 1730 KN

1 kip = 4.448 KN
1 in. = 25.4 mm

Analytical Background

The differential equation for the laterally loaded pier/pile problem, as derived from conventional beam theory (Hetenyi, 1946), is:

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} - p = 0 \quad \text{---------(1)}$$

where

EI = flexural rigidity of pier
y = deflection of pier
x = length along pier
P_x = axial load
p = soil reaction per unit length.

The differential equation is solved by difference equations formulation using a computer program such as LPILE OR PILED/G. The soil behavior is modeled as p-y curves representing soil resistance vs pier deflection. The p-y curves describing the soil response are nonlinear and depend on several parameters, including depth, unit weight, shearing strength and stress-deformation characteristics of the soil.

Development of p-y curves

LPILE

The most commonly used criteria for development of p-y curves in sands are proposed by Reese et al.(1974). We used this sand p-y formulation for modeling the test piers in LPILE. The criteria were developed from the results of lateral load tests on a 0.61-m (24-inch) diameter flexible, instrumented pipe pile embedded in a deposit of submerged, dense fine sand. Both theory and empirical factors were used to obtain mathematical expressions that fit the experimentally derived p-y curves. The primary input parameters controlling the lateral soil stiffness in the model are soil friction angle and coefficient of subgrade reaction (k-value). Water table effects are modeled by using buoyant unit weight for computing ultimate passive resistance in the model.

PILED/G

For lateral load analysis of the test piers using PILED/G, the sand p-y formulation proposed by Bhushan (1981) was used. The primary parameter controlling the lateral stiffness in this model is relative density. Water table effects are modeled by using a p-multiplier of 0.5 below the water table. The PILED/G sand procedure for developing p-y curves is described in the following sections.

For solving equation (1), a secant modulus of soil reaction, E_s, is defined as:

$E_s =$ p/y .... (2)

For sands, E_s is generally assumed to vary linearly with depth such that

$E_s =$ Kx .... (3)

where

K = constant relating the secant modulus of soil reaction to depth ($E_s = Kx$), KN/m^3 (pci).

Assuming that E_s increases linearly with depth, using equations (2) and (3):

p = Kxy .... (4)

where

p = soil reaction per unit length, KN/m (lb/in.)
x = depth at which p-y curve is defined, m (in.)
y = lateral deflection at depth x, m (in.)

Major factors affecting K are relative density of the sand and lateral deflection. Based on the procedure suggested by Bhushan and others (1981), the relationship shown in Table 2 for variation of K with y/D (deflection, y normalized with respect to the pier diameter, D) is adopted in PILED/G. This variation of K with normalized deflection is applicable to dense sands with a relative density of about 85 percent.

Based on the relationship suggested by Meyer and Reese (1979), A simplified function shown in Table 2 to obtain the K values for relative densities other than 85 percent is used in PILED/G.

The p-y curves in sand are developed as follows:

a. From blow count data, cone penetrometer tests, pressuremeter tests, or other available data, estimate the relative density of the and deposit.

b. From laboratory tests or available correlations, estimate the angle of shearing resistance, ϕ, and soil unit weight, γ.

c. For p-y curve at any depth x and for a pier with diameter D, compute a set of values of lateral deflection, $y_i = y_1, y_2\ y_3,...,y_9$ corresponding to $(y/D)_i$ values in Table 2 by:

   $y_i =$ $(y/D)_i$ (D) ....(5)

d. Compute corresponding values of soil resistance, p, by:

   $p_i =$ $(K_i)(x)(y_i)(F1)(F2)(F3)$ ....(6)

   in which

   $y_I =$ $y_1, y_2..y_9$ are values of y given by equation (5)
   $K_I =$ $K_1, K_2..K_9$ are corresponding values of K from Table 2.1.
   $F_1 =$ density factor from Table 2.2
   $F_2 =$ slope factor
   $F_3 =$ groundwater factor -- Use $F_3 = 0.5$ Below the groundwater, and $F_3 = 1.0$ Above the groundwater

Selection of Soil Parameters

The residual/formational materials in which the load tests were performed are dense, overconsolidated, granular soils with some degree of cementation or interlocking which provides an insitu cohesion component. Once excavated, the materials are silty to clayey sands (the conglomerate also contains significant gravel and cobbles). These materials may be modeled as a cohesionless material, but due to overconsolidation behave much stiffer than normal dense sand.

TABLE 2 VARIATION OF K WITH DEFLECTION AND RELATIVE DENSITY

Table 2.1

i	$(y/D)_i$, mm/mm	$K_i(D_r$=85%), pci
1	0.0000	0
2	0.0010	560
3	0.0025	390
4	0.0050	260
5	0.0100	140
6	0.0166	90
7	0.0250	65
8	0.0750	30

Table 2.2

Relative Density D_r, percent	Factor F1
100	1.250
85	1.000
70	0.750
50	0.400
35	0.125

i = (y/D) and K index
y = Pier/Pile deflection at groundline, mm.
D = Pier/Pile diameter, in.
K = Constant relating the secant modulus of soil reaction to depth (E_s=Kx), lb per cubic in.
D_r = Relative density of sand, percent.
F1 = Relative density factor for D_r other than 85%.

NOTES:

1. Values of K given in Table 2.1 are applicable for a relative density of 85%.
2. For relative densities other than 85%, the values of K in Table 2.1 should be multiplied by the density factor F1, shown in Table 2.2.
3. Linear interpolation should be used for values not in the table.
4. 1pci = 271.4 KPa /m

The selected soil parameters for analysis of the test piles with the two p-y curve models are discussed in the following sections.

LPILE

For use with the program LPILE (ENSOFT, 1997), we used the Reese sand model (Reese et el., 1974) with a friction angle of 42 degrees and a coefficient of subgrade reaction (K value) of 61 MN/m^3(225 pci) for our initial predictions. This is the maximum k-value recommended in the LPILE manual for dense sand. The observed and predicted load deflection curves are presented in Figures 3 through 5.

PILED/G

For use with the program PILED/G (GEOSOFT, 1988), we used the sand model (Bhushan, 1981). In this model, the primary parameter that defines the soil behavior is the relative density. We used a relative density of 100%. For dense formational materials the PILED/G manual recommends use of a p-multiplier of 1.25 to 2.0. We used a p-multiplier of 1.5 for our initial predictions. The observed and predicted load deflection curves are presented in Figures 3 through 5.

COMPARISON OF OBSERVED AND PREDICTED BEHAVIOR

A discussion of the observed and predicted behavior is provided for each type of formational material tested (see Figures 3 through 5 and Table 1).

Decomposed Granite

The observed values of deflection for loads up to 890 KN (200 kip) were less than 4 mm (0.16 inch). The predicted values using the PILED/G program with 100% relative density and p-multiplier of 1.5 are 2.5 to 8.0 times the observed values. For LPILE without p-multipliers, the predicted values are 30 to 50 times the observed values. The calculated load for 25-mm (1-in.) deflection using LPILE is about 300 KN (70 kip) and using PILED/G is about 890 KN (200 kip). Although the test piers were only loaded to 850 KN (190 kip) load due to the limitations of the loading equipment, extrapolation of the data indicates that loading for 25 mm (1 in.) deflection could be on the order of 1500 to 2000 KN (335 to 450 kip). This estimated load is four to six times the value predicted by LPILE without p-multipliers and about 1.5 to 2.25 the value predicted by PILED/G without p-multipliers.

Stadium Conglomerate

For the 0.81-m (32 inch) average diameter piers, the measured values of deflection for the maximum load of 1780 KN (400 kip) ranged between 28 and 48 mm (1.1 and 1.9 inches). The predicted deflections using the PILED/G program without p-multipliers are between 1.2 and 2.2 times the observed values over the range of loads. For LPILE, the predicted deflections without p-multipliers are between 3.2 and 7.8 times the measured values. The ratios of the measured to the predicted load at 6.4 and 25.4 mm (0.25 and 1 in.) deflection using LPILE are 2.3, and 2.3 to 2.8, respectively. The corresponding ratios using PILED/G are 1.2, and 1.3 to 1.5, respectively.

Friars Formation

For the 0.61-m (24 inch) average diameter piers, the observed values of deflection for the maximum load of 1780 KN (400 kip) ranged between 43 mm and 71 mm (1.7 and 2.8 inches). At this load, without p-multipliers PILED/G over-predicted the deflection by a factor of 3.4 to 5.6, and LPILE over-predicted the deflection by a factor of 10 to 20. The ratios of the measured to the predicted load at 6.4 and 25.4 mm deflection using LPILE without p-multipliers are 3.7 to 4.9, and 4.2 to 4.8, respectively. The corresponding ratios using PILED/G are 1.9 to 2.5, and 2.3 to 2.7, respectively.

Parameters Required to Match Observed Behavior

Using the PILED/G sand model with 100% relative density, p-multipliers of 3.0, 2.0, and 5.0 are required to match the observed behavior in DG, Stadium Conglomerate, and Friars Formation, respectively. Using the LPILE sand model with a friction angle of 42 degrees and K = 61 MN/m^3 (225 pci), the corresponding p-multipliers are 8.0, 3.5, and 8.0, respectively. The observed and predicted values using these p-multipliers are shown in Figures 3 through 5. This comparison indicates that, in general, using dense sand parameters, LPILE requires p-multipliers about twice as large as PILED/G.

CONCLUSIONS

The results of the lateral load tests indicate the following.

For 0.6-m to 0.8-m (24 to 32-inch) diameter drilled piers with a penetration of about 6 m (20 ft) in Stadium Conglomerate and Friars Formation the following results are obtained:

1. At a specified lateral deflection of 6.4 mm (0.25 in.), the mobilized lateral load is about 500 KN to 660 KN (112 to 148 kip).

2. At a specified deflection of 25 mm (1 in.), the mobilized lateral load is about 1300 KN to 1685 KN (292 kip to 379 kip).

3. The “ultimate” lateral load corresponding to a deflection of about 50 mm (2 in.), ranges between 1680 and 1770 KN (378 and 398 kip).

4. For comparison, the “ultimate” lateral load for similar piers using p-y curves for very dense sand would range between 445 and 1335 KN (100 kip and 300 kip).

For 1-m (3-ft) diameter and 3-m (10 ft) deep piers in DG, the following results are obtained:

1. The projected load at 6.4-mm (0.25 in.) lateral deflection is on the order of 1250 KN (280 kip).

2. The ultimate load for this pier is projected to be greater than 2000 KN (450 kip).

3. For comparison, the “ultimate” lateral load for similar piers in very dense sand would range between 445 and 890 KN (100 kip and 200 kip).

These data indicate that the residual/formational materials tested have ultimate lateral capacities of 2.0 to 2.5 times the lateral capacity of similar piers in dense sand. Consequently, use of dense sand parameters for design of drilled piers in these soils will result in over-prediction of deflections and larger penetrations than required. Pier design based on appropriate p-multipliers of 2 to 5 for PILED/G and 3 to 8 for LPILE, can result in significantly shorter piers, savings in foundation costs, and more accurate dynamic modeling of the superstructure.

ACKNOWLEDGMENTS

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