FIELD EXPERIMENTAL STUDY OF LOAD-DEFLECTION BEHAVIOR OF DRIVEN PILES IN SOFT BANGKOK CLAY

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ABSTRACT

This paper presents the details and preliminary results of full-scale lateral load tests on driven piles in Bangkok clay. Close-ended steel pipe piles of moderate size were installed into soft Bangkok clay. Strain gauges were attached on the surfaces of the piles at various locations along their length. The strain gauge outputs were employed to determine pile bending moments, deflections and external soil reactions. Lateral loads up to 10 ton were applied to these test piles. Maximum head displacements between 98-115 mm were observed. The locations of maximum moments shifted to greater depths with increasing lateral load, but they never exceeded a depth of 12B below the ground surface. The computer program LPILE with Matlock's p-y model for soft clay over-predicted the actual load-head displacement relations. The Characteristic Load Method, however, predicted overly conservative values of head displacement, particularly for large magnitudes of lateral load.

KEYWORDS: Full-scale load test, Lateral load, Driven pile, Pile response, Bangkok clay

1. Introduction

Piles are a commonly used type of deep foundations for several structures such as bridges, highways and high-rise buildings. They are deeply embedded into foundation soils to support the loads of superstructures in both vertical and horizontal directions. Lateral loading of piles is one major class of pile analyses which involves the interaction between soil and structure. The response of laterally loaded piles is influenced by the combined effects of the pile flexural stiffness and stress-strain and strength characteristics of the foundation soils. A better understanding of pile-soil interaction is therefore imperative for proper analysis and design of laterally loaded piles.

Broms [1] introduced an analytical method to determine the safe embedded lengths of short rigid piles and long piles with plastic hinges. This method considers passive and active earth pressures acting on different parts of the pile as it rotates under lateral loading. Hetenyi [2] presented an approach to analyze the load-deflection behavior of laterally loaded piles based on their analogy to an elastic beam on foundation. In this method, the response of foundation soils to the deflection of pile is represented by non-linear p-y relationships. Along with this process, several p-y models have been proposed for different soil types [3, 4 and 5]. Evans and Duncan [6] developed a simplified method based on the results of numerical parametric simulations of laterally loaded piles with various p-y models. Several design charts have been proposed to determine the head deflection and maximum bending moment of pile for different fixity conditions at pile head. In recent years, the finite element method has increasingly been employed to analyze the behavior of laterally loaded piles [7]. The pile and foundation soils are modeled as three-dimensional deformable bodies. Their mechanical response and the behavior at the interface between the pile and soil are represented by proper constitutive laws.

Full-scale load testing serves as a means of paramount importance not only to better understand the actual pile behavior under lateral loads but also to validate the results of the aforementioned analytical and numerical methods. However, a very limited number of lateral load tests on piles or drilled shafts in Bangkok clay have been performed. As such, full details of the load-deflection behavior of deep foundations in this particular urban soil are not available. The present study was undertaken to fill such a gap. Field load tests on instrumented steel piles were performed to investigate the load-deflection characteristics of piles up to large pile displacements.

2. Site information and properties of foundation soils

Full-scale lateral load tests were performed to study the load-deflection response of driven piles in Bangkok clay. The site for these field load tests was located in Mahidol University, Nakhon Pathom, Thailand. A subsurface exploration program was conducted to characterize the ground conditions. Two boreholes 20 m deep were drilled by wash boring. Intact samples were collected every 1.5 m using Shelby tubes from a depth of 1.5 m to 13.5

m. At greater depths, the soils were relatively stiff, and samples were obtained by split-barrel samplers. Standard penetration tests were performed every 1.5 m from a depth of 13.5 m towards the ends of boreholes.

The groundwater level was observed to be at about 1.0 m below the ground surface. Laboratory index tests, one-dimensional consolidation tests, unconfined-compression tests, unconsolidated-undrained (UU) triaxial tests and one consolidated-undrained triaxial test (CU) were performed. From the ground surface to a depth of 12 m, the soil deposit consisted mainly of soft-to-medium clay of high plasticity. There was, however, a seam of low-plastic clay between 4-5.5 m. The top clay layer was slightly overconsolidated, and the overconsolidation ratios)OCR (varied from 2.5 at a depth of 3.25 m to 2.0 at 7.75 m .The undrained shear strengths determined by the laboratory UU and CU triaxial tests were in good agreement. They appeared not to change appreciably with depth and consistently varied between 25-35 kPa. However, the undrained shear strengths obtained from the UC tests were slightly larger. This is likely because the UC tests were performed earlier in the experimental program, and the developed negative pore pressures in the soil specimens due sampling had not yet significantly dissipated. For the UU tests, which were performed later, the negative pore pressures prior to testing had further dissipated. Although positive cell pressures were applied during undrained shear, the corresponding effective stresses inside the UU specimens were relatively less, and this resulted in smaller undrained shear strengths as reported. For the CU triaxial test, the specimen was obtained at 7.75 m below the ground surface. To simulate the field conditions, the triaxial specimen was consolidated to a maximum isotropic effective stress of 105 kPa and then rebounded to 51 kPa, thus resulting in an OCR of 2.06 prior to shear.

The top clay layer was underlain by a layer of very stiff clay of high plasticity 6 m in thickness (12-18 m). The values of N-SPT determined at various depths within this layer were almost similar, and these were close to 20. For depths of 18-20 m, a layer of low-plastic clay was encountered. The soil at these depths was much stiffer, and N-values above 50 were observed. The results of exploratory borings and laboratory experiments to characterize the ground conditions are summarized in Table 1 and Figure 1.

Table 1 Soil layers and their engineering properties

Depth (m)	USCS Symbol	γ (kN/m^3)	ω (%)	OCR	LL (%)	PI (%)	Undrained shear strength (kPa)			N-SPT (Blows/ft)
							UC	UU	CU	
0.0-4.0	CH	16.3-16.6	42-60	-	50-55	30-31	32-33	32-35	-	-
4.0-5.5	CL	16.6-16.7	47-58	2.54 (3.25 m)	40-41	21-25	29-30	28-30	-	-
5.5-12.0	CH	15.6-16.2	47-65	2.06 (7.75 m)	50-68	28-40	26-35	26-32	29-30	-
12.0-13.0	CH	19.4-19.5	26-28	5.17 (12.25m)	59-60	35-37	>100	-	-	-
13.0-18.0	CH	-	23-26	-	50-60	33-42	-	-	-	22-35
18.0-19.0	CL	-	21-22	-	48-49	32-33	-	-	-	35-40
19.0-20.0	CL	-	20-21	-	28-30	11-12	-	-	-	40-50

Note: γ is the total unit weight (kN/m^3), ω is the natural water content (%), OCR is the overconsolidation ratio, LL is the liquid limit of soil (%), PI is the plasticity index (%) and N-SPT is the number of blows according to the standard penetration test (Blow/ft).

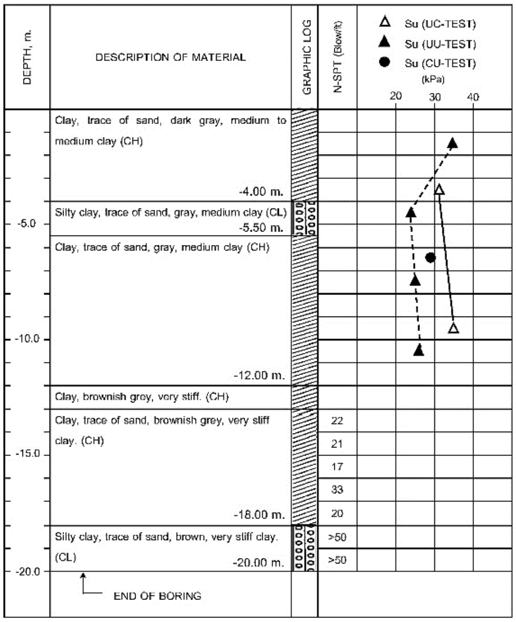

Figure 1 Profile of foundation soils at test site and relevant strength parameters

Other relevant areas

3. Pile instrumentation and test setup

The experimental program consisted of three lateral load tests on identical pipe piles. The test piles were 12.5 m long, 0.25 m in inner diameter and 9 mm thick. Conical protective shoes 0.5 m long were fitted to the bottom ends to prevent the piles from being damaged during driving. The pipe material conformed to the specifications of ASTM A53 standard [8]. Strain gauges were attached, in pair, on the opposite sides of piles to monitor the flexural response. The strain gauges were integrated into the data acquisition system using a quarter-bridge type of circuitry .Four strain gauges were therefore used at one pile depth .This arrangement allowed for measurement of the deflections of pile under both compression and tension and ensured the availability of output signals even when some strain gauges might be damaged. Strain gauges were installed every 0.5 m from the ground surface to a depth of 4 m. At greater depths where small deflections were expected, strain gauges were located further apart, i.e.; every 1 m for depths of 4 to 7 m and every 2 m thereafter.Two steel C-shaped channels were attached to the piles, thus covering the strain gauges and serving as protective cases during pile driving. A separate calibration bending test was performed on an instrumented section of test pile to establish the bending moment-curvature relationships. The test results indicated that the yield bending moment of test pile was 210 kN-m, and the “elastic” flexural stiffness prior to yield was about 21,000 kN-m^2.

Pile driving was carried out using a 5.2 ton drop hammer. All piles were installed to a depth of 12 m. In total, seven reaction piles were employed. The test piles and corresponding reaction piles were located 2.2 m apart so as to accommodate a horizontal load - providing assemblage as shown in Figure 2a. A 40 ton load cell was employed to monitor the lateral loads applied to the pile head. The head displacements of piles were tracked by three LVDTs attached to the piles at different heights above the ground surface. The output signals from the strain gauges, load cell and LVDTs were recorded and processed using a dedicated data acquisition computer. The setup for pile load tests is shown in Figure 2b.

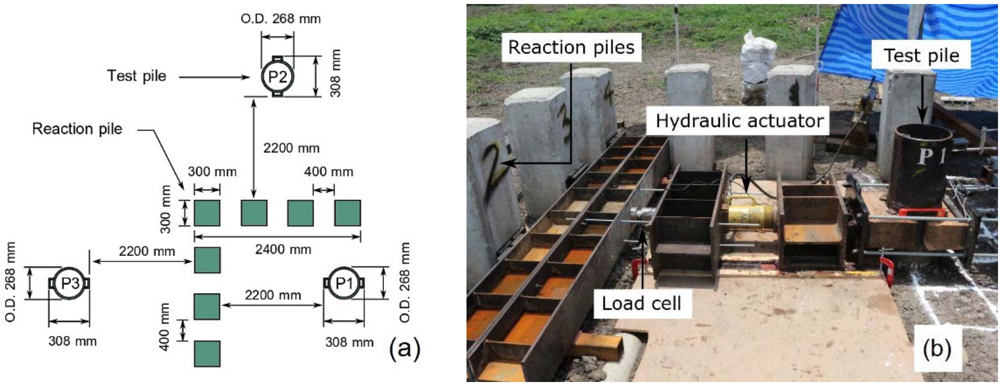

Figure 2 Arrangement of test and reaction piles (a) and load-providing assemblage (b)

4. Full-scale lateral load tests

Lateral loads were applied in increments of 1 ton until maximum loads close to 10 ton were attained. In each increment, the hydraulic actuator was extended at rates close to 1 mm/min until the target load was achieved. This rate was identical to that used to determine the undrained shear strengths of foundation soils from the laboratory UU triaxial tests. This protocol for load application complied with the specifications of ASTM D3966-07 standard [9]. As shown in Figure 3, the applied lateral load was held constant for 15 minutes for each step. This was to allow the interaction between pile and soils to reach equilibrium before the next load step was commenced.

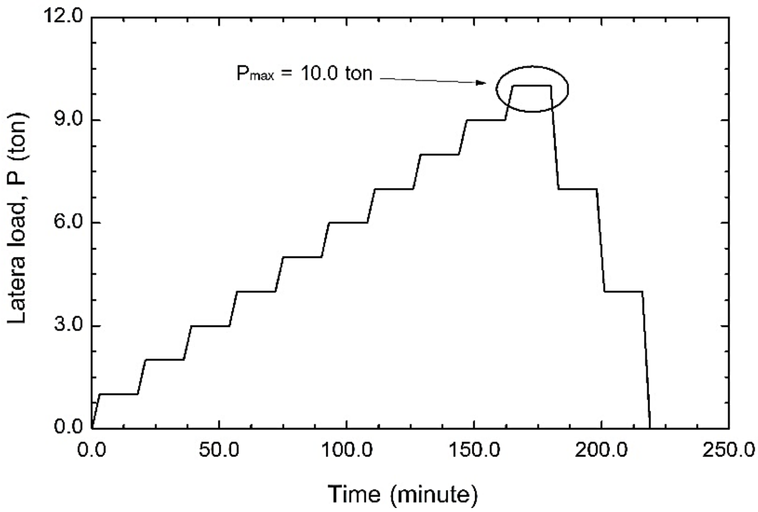

Figure 3 Protocol for load application

Other relevant areas

An inclinometer was also utilized to monitor pile lateral deflections. Inclinometer casings 12 m long were attached to the insides of test piles using welded U-bolts. High-strength epoxy was used to fix the casings and bolts together, thus preventing slippage of the casings during pile installation and testing. Inclinometer readings were recorded manually every 0.5 m for the entire depth of test piles.

5. Pile response

The strain gauge results were analyzed within the framework of elastic beam on foundation. The pile under consideration is assumed to be elastic with a constant value of flexural stiffness (EI). The curvature ($\emptyset$) is expressed in form of $\emptyset = 2\varepsilon/B$ where ε is the longitudinal strain on the surface of pile and B is the pile diameter.

The bending moment is computed as $M = EI\emptyset$ The shear force (V) and perpendicular external load or soil reaction (p) in this application can be determined by differentiating the bending moment once and twice, i.e.; $V = dM/dz$ and $p = dV/dz = d^2M/dz^2$. The pile slope (S) and deflection (y), on the other hand, are computed from integration of the curvature, i.e.; $S = \int \emptyset dz = (1/EI) \int M\, dz$ and $y = \int S\, dz = (1/EI) \iint (M\, dz)dz$. In these formulations, the variable z denotes the depth or pile coordinate.

The variations in strain (ε) obtained from the strain gauge readings were used to compute the curvatures and bending moments at various pile depths. The pile slopes and deflections were determined by approximating the analytical integration using a trapezoidal rule:

$$\int f\, dz \approx \sum_{i=1}^{n} \left(\frac{f_i + f_{i-1}}{2}\right)(z_i - z_{i-1}) \qquad (1)$$

where n is the number of strain gauge elevations. The shear forces and soil reactions were calculated from the bending moments using a two-point finite difference formula:

$$\frac{df}{dz} = \frac{f_{i+1} - f_{i-1}}{z_{i+1} - z_{i-1}} \qquad (2)$$

It was observed that the experimental maximum bending moments in piles were less than the yield bending moment of 210 kN-m for the applied maximum lateral loads. For such magnitudes of lateral load, the relationship between the pile bending moment (M) and curvature (∅) is still linear; the value of EI is constant and the calculations of S, y, V and p using the presented numerical differentiation and integration were performed accordingly.

The typical variations of pile bending moment (M), pile deflection (y) and soil resistance (p) of load steps of 4, 7 and 10 ton obtained, as examples, from the first load test are plotted versus depth in Figure 4. It can be observed that the maximum bending moments increase and take place at slightly greater depths with increasing P. The locations of maximum bending moments are within 2.5-3.0 m or 10-12B below the ground surface. This depth is greater than the upper limit of 8-10B usually reported in the literature [10].

The maximum bending moment of pile for the maximum lateral load (P = 10 ton) is 195 kPa. Near the ground surface, the test pile deflects in the direction of lateral loads (Figure 4b). The trend, however, reverses and the pile deflects in the opposite direction at greater depths. The magnitude of pile deflection, not surprisingly, increases with P. The variation of soil resistance with depth generally follows the trend of bending moment. The locations of maximum soil reactions, however, take place at slightly greater depths compared to those of the maximum bending moments.

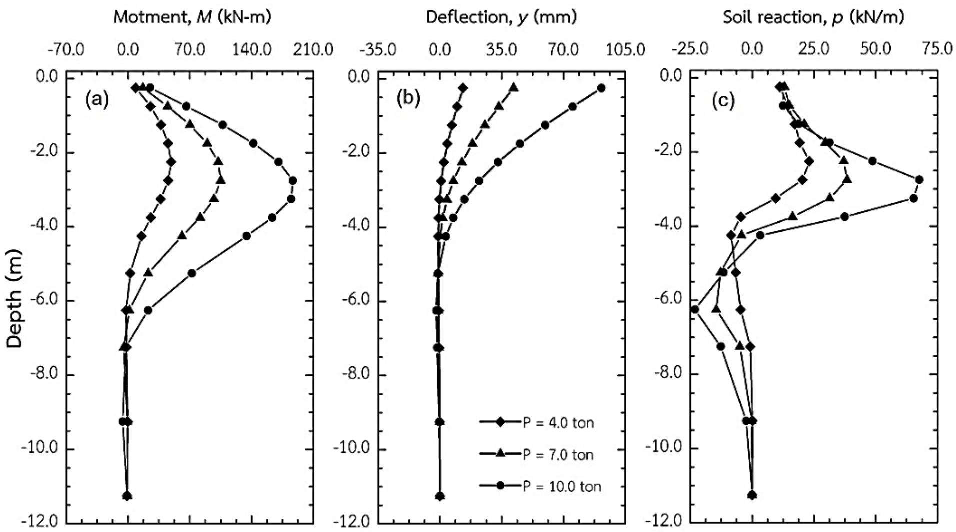

Figure 4 Moments (M), pile displacements (y) and soil reactions (p) for different lateral loads.

The experimental load-head displacement relations are shown in Figure 5. The curves suggest stiff load- displacement response of pile at small displacements. At large displacements, the curve continues to gradually increase, however, at smaller rates. When P equals 10 ton, the corresponding maximum head displacements vary between 98-115 mm. The head displacements obtained from the three tests vary within 15% of each other. Such experimental scatter is fairly small, and it suggests that the adopted testing methodology can produce repeatable experimental results.

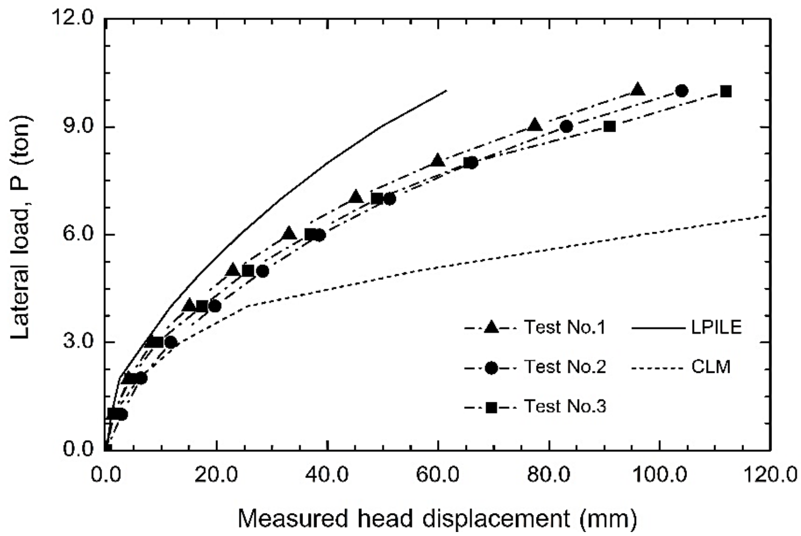

Figure 5 Comparison of actual load-head displacement relationships and predictions of LPILE and Characteristic Load Method

The experimental load-head displacement curves are compared to the numerical predictions of the computer program LPILE and the results of the Characteristic Load Method (CLM) proposed by [6] and [10]. For LPILE, the pile is discretized into elements and each is attached to a Winkler spring in the perpendicular direction. The spring response is characterized by non-linear p-y relations of user's choice.

The accuracy of this type of numerical analysis depends on the ability of the employed p-y model to predict the actual pile-soil behavior. Matlock's p-y model for soft clay was used in the LPILE analysis. This particular p-y model is widely used within the local geotechnical engineering community to analyze the load-deflection behavior of piles and drilled shafts in Bangkok clay. It can be observed that LPILE significantly over-predicts the experimental curves. In other words, the predicted head displacements are less than the actual ones, thus being not preferable from a practical perspective. The difference between numerical and experimental results increases with the magnitude of P. Such a difference between the experimental and numerical results is likely because Matlock's p-y model assumes an unrealistic failure mechanism of the soil adjacent to the back of the top portion of pile. This results in predictions of excessive ultimate resistance of soil and, therefore, too stiff p-y relationships. The predicted stiff p-y response results in unrealistically small head displacements as reported.

The Characteristic Load Method is developed based on numerical parametric simulations of laterally loaded piles in different soil types. Such numerical analysis is also performed within the framework of pile attached to Winkler's springs. The results are compiled in form of design charts, from which the head displacement can be determined by entering the value of applied lateral load normalized by the characteristics shear force into the proper chart. In the present study, a pile in clay subject to free-head condition is assumed. The corresponding head displacements are then determined for different lateral loads applied at the pile head. It can be observed that the results of Characteristic Load Method plot slightly below the experimental curves for lateral loads smaller than 4 ton. At higher P values, however, it significantly under-predicts the actual load–head displacement relationships.

It may thus be concluded that the use of Characteristic Load Method in practice can result in overly conservative predictions of head displacements of piles subject to free-head conditions, particularly at high magnitudes of P. The computer program LPILE with Matlock's p-y model for soft clay may also be used to predict the pile behavior. However, proper multipliers or reduction factors should be employed so as to obtain softer p-y response and therefore more realistic predictions of the load-head displacement relationships.

6. Repeatability and accuracy of field test results

The variations of bending moment, pile deflection and soil reaction of the load step of 3 ton for the three load tests are shown in Figure 6. These typical M, y and p curves of all tests plot fairly close to each other. Experimental scatter less than 10% can be observed. Such magnitudes of scatter among these load tests do not noticeably change when different load steps are compared. This implies that the present experimental procedure and the results so obtained are of high repeatability.

The pile deflections directly measured by inclinometer are also plotted in Fig. 6b. The deflection curves determined from the strain-gauge readings and that of the inclinometer measurements are in good agreement. This trend also holds for other load steps. This suggests that the values of y presented herein are accurate and of high reliability.

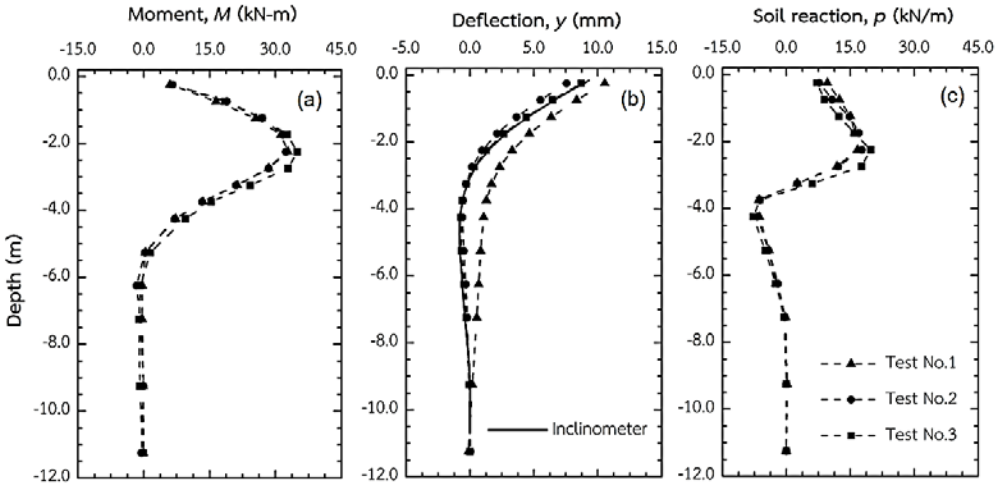

Figure 6 Variability of load-deflection characteristics for P = 3 ton

7. Conclusions

The purpose of this study was to determine the load-deflection behavior of laterally loaded piles in Bangkok clay. Three load tests on identical driven piles in this urban soil were performed. The results indicated that the variability of bending moments (M), pile deflections (y) and soil reactions (p) among the three load tests was fairly small. Experimental scatter less than 10% was observed. The maximum bending moments increased with increasing lateral load (P). The locations of maximum bending moments were within a depth of 2.5-3.0 m or 10-12B below the ground surface. It was observed that the computer program LPILE significantly over-predicted the experimental load-head displacement curves. However, the Characteristic Load Method under-predicted the actual load-head displacement relationships, particularly for lateral loads greater than 4 ton. It was thus concluded that the use of Characteristic Load Method in practice can result in overly conservative predictions of head displacements of piles subject to free-head conditions. The computer program LPILE with Matlock's p-y model for soft clay might also be used. However, proper multipliers or reduction factors should be employed so as to obtain softer p-y response and therefore more realistic predictions of the load-head displacement relationships of laterally loaded piles in soft Bangkok clay.

Acknowledgement

The study presented herein was funded by the Thailand Research Fund (TRF) and SEAFCO Public Company Limited. Such financial supports are gratefully acknowledged. The authors wish to thank all members of the Geotechnical Engineering Laboratory at Mahidol University for their assistance during the course of field load tests.

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Topics of interest: Deformation and stability of geostructures, soil-structure interaction, railway geotechnology, mechanical behavior of geomaterials and finite element techniques.