10th International Science, Social science, Engineering and Energy Conference
20th-23rd November, 2019, Rajamangala University of Technology Isan Sakon Nakhon Campus, Thailand

I-SEEC 2019

http://i-seec2019.rmuti.ac.th

A Hyperbolic P-Y Model for Analysis of Laterally Loaded Piles in Bangkok Clay

Bhashkar Pathak [1-a] and Pongpipat Anantanasakul [1-b]

[1] *Department of Civil and Environment Engineering, Faculty of Engineering, Mahidol University, Nakhon Pathom 73710, Thailand*

[a]*bhashkar.pathak11@gmail.com*, [b]*pongpipat.ana@mahidol.ac.th*

Abstract

P-y models are extensively used by practicing engineers to analyse and design laterally loaded piles. Existing *p-y* models for soft clay, medium clay, and stiff clay predict unrealistically stiff load-deflection behavior of piles in Bangkok clay. A full-scale lateral load test was performed on a driven steel pipe pile 12.5 m long and 0.268 m diameter. The pile that was installed in the upper soft-to-medium layer of Bangkok clay was instrumented with strain gauges so as to sense the deflections in response to the applied lateral loads at the pile head. The load-deflection characteristics of the pile were later analyzed from the strain-gauge readings based on the framework of beam on elastic foundation. This allowed for determination of experimental $p - y$ relationships for the test pile in Bangkok clay. The present study proceeds to propose a $p - y$ model based on the field experimental results. It has been observed that the proposed $p - y$ model can back-predict the observed lateral load-deflection behavior of the test pile in Bangkok clay with excellent accuracies.

Keywords*: p-y model, lateral load test, pile foundation, Bangkok clay, load-deflection behavior*

1. Introduction

Pile foundations are extensively used for high rise buildings, towers and bridge abutments. In general horizontal forces on foundations arise due to wind loads, earthquake loads, eccentric loads and waves among others. It has been observed that the response of a laterally loaded pile is a combined effect of 1) the flexural stiffness of the pile, 2) the stress-strain and strength behaviour of the foundation soil and 3) interaction of these two factors [1,2,3]. Such a combined response is generally non-linear, and this results in an increase in complexity when the load-deflection characteristics of piles under lateral loads are analysed. Several means have been proposed to aid in analysis and design of laterally loaded piles. The linearly elastic solution presented by Poulos and Davis [4] emphasizes the condition of continuity although the soil is unrealistically characterized as a linearly elastic material. The

limit equilibrium solution proposed by Broms [5] can be applied in finding the ultimate lateral load at failure, but the soil-structure interaction at smaller loads is not addressed.

Hetenyi [6] presented a practical approach to analyse the deformational responses of laterally loaded piles based on their analogy to a beam on elastic foundation. Analytical solutions of the differential governing equation have been introduced for some simple boundary conditions [7]. In this framework the response of the foundation soil to the deflection of pile is represented by a non-linear p-y relationship. Computer codes based on the finite difference method, such as LPILE [8], have been developed to solve the pile governing equation for more realistic boundary conditions. Along with this process, several p-y models have been proposed for different soil types [9,10,11,12]. *P-y* models for cohesive soils usually take the forms of simple mathematical functions such as parabolic and hyperbolic equations [13]. The initial part of a $p - y$ curve is controlled by the stiffness while the ultimate soil resistance governs the final portion.

Existing $p - y$ models for clays [9,11,12] have been routinely used by local foundation engineers for analysis and design of deep foundations under lateral loading in Bangkok clay. When compared to the results of conventional lateral load tests, the accuracy of the existing $p - y$ models has been observed to be acceptable only when the piles undergo small head deflections. For large lateral forces and the corresponding deflections, discrepancy among the numerical predictions and the test data noticeably increases. This suggests that the existing $p - y$ models may not be the most accurate tools for this particular urban soil.

To address this issue, Chaosittichai and Anantanasakul [13,14,15] performed a lateral load test on an instrumented pipe pile in Bangkok clay. The driven steel pile 12 m long and 0.268 m in diameter was loaded to a very large head displacement of 125 mm, thus well beyond soil failure. The variations of pile deflection with depth were carefully analysed in response to different applied loads. In consequence the present study was undertaken so as to establish an accurate and reliable $p - y$ model from this set of quality load-test results. In this paper the full-scale lateral load test program and the method used to analyze the experimental data are reviewed. The development of the $p - y$ model is discussed and its capability to back-simulate the load-deflection behavior of the test pile is assessed.

2. Full-scale lateral load test

2.1 Site information and subsurface conditions

The site for this full-scale load test is located about 30 km northwest of Downtown Bangkok. It is part of Mahidol University - Salaya Campus as shown in Figure 1. The elevation of this site is about 2 m above mean sea level. The subsurface conditions were

obtained from two boreholes 20 m deep; one of which was drilled at the centre of the test site, while the other was located 50 m away. Groundwater level was observed to be at 1 m below the ground surface during the course of field testing. Intact samples were collected every 1.5 m from a depth of 1.5 m to 13.5 m using Shelby tubes. At greater depths, the soils were relatively stiffer, and samples were routinely retrieved by split- barrel samplers as part of the standard penetration test (SPT).

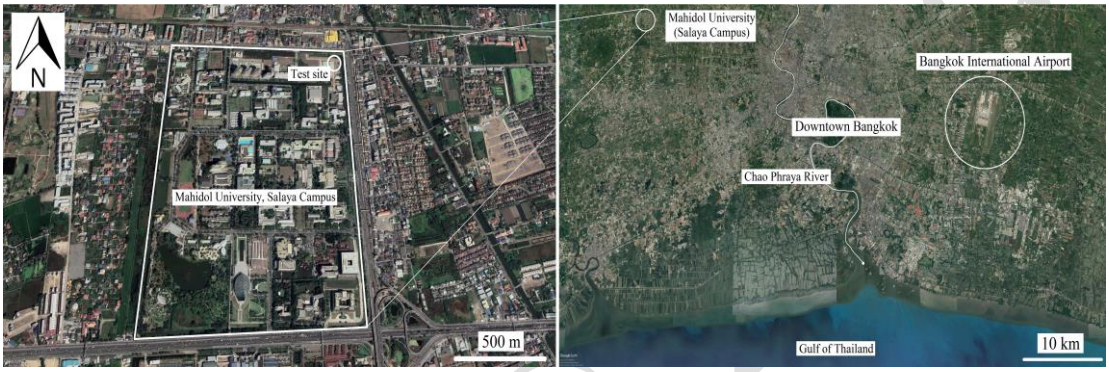


Figure 1 Test site located 30 km northwest of Downtown Bangkok and is part of Mahidol University – Salaya Campus.

Relevant mechanical properties of the foundation soil as obtained from laboratory and field experiments are reported in form of a boring log in Figure 2. The values of shear strength vary somewhat between 25-35 kPa for the upper 12 m from the ground surface, and SPT values close to 25 can be observed from 12 m to 20 m. Two key sectors of foundation soils can be identified based on the strength and SPT data; a layer of relatively uniform soft-to-medium clay of high plasticity 12 m in thickness underlain by a sequence of stiff to hard silty clay layers (CH and CL) towards the ends of boreholes. There is, however, a seam of low-plastic silty clay 1.5 m thick from 4 m to 5.5 m. One-dimensional consolidation tests were also performed and the results indicate that the upper clay layer is slightly overconsolidated with OCRs varied from 2.5 at 3.25 m to 2 at 7.75 m. It is noteworthy that the subsurface conditions for this test site conform to the typical trend of that of the Greater Bangkok area as reported by Horpibulsuk et al [16].

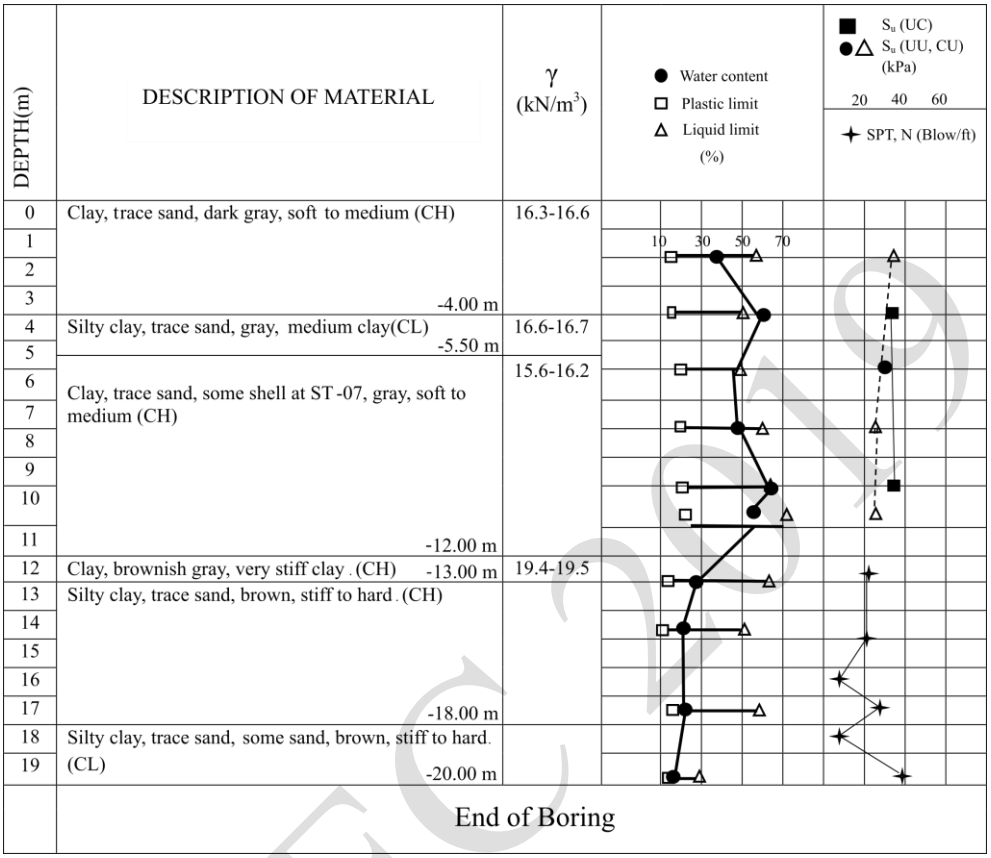
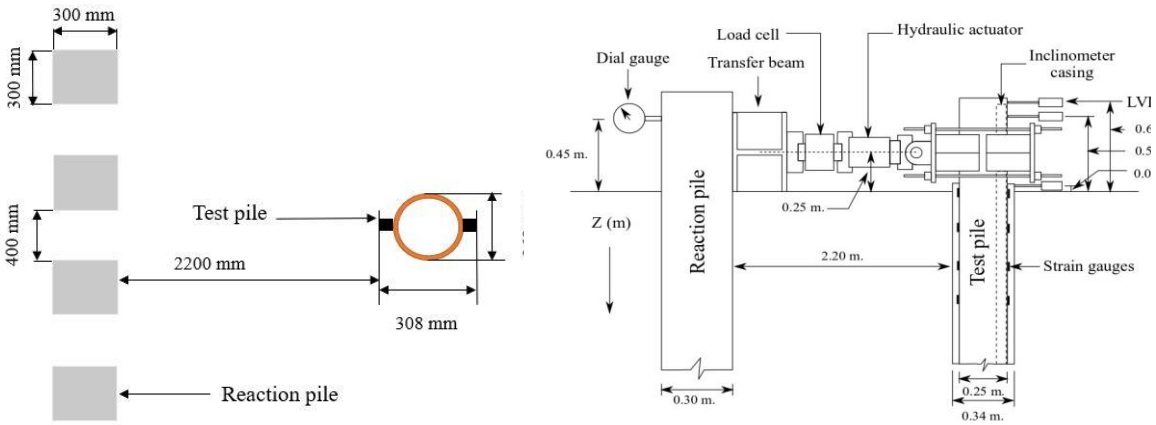


Figure 2 Boring log, strength and SPT profiles and relevant properties of foundation soils.

2.2 Arrangement of field load test

The full-scale lateral load test [14,15,16] was performed on a steel pipe pile 12 m long, 0.268 m in outer diameter and 9 mm thick. Four concrete piles 13 m long were used as reaction piles (Figure 3a). The test and reaction piles were driven using a 5.2 ton drop hammer. The test pile was driven, closed-ended, to a depth of 12 m, while the reaction piles were installed to a depth of 13 m to form a 4x1 group. The centre-to-centre spacing among the concrete piles was 0.7 m, thus rendering a width of 2.4 m for the group. The test pile and the reaction piles were located 2.2 m apart to accommodate the horizontal load-providing assemblage as shown in Figure 3b.



(a) (b)

Figure 3 Arrangement of test pile and reaction piles (a) and schematic diagram of load providing assemblage (b).

This assemblage consisted of a hydraulic actuator, load cell, and transfer beams. A pin support was used at the connection between the test pile and the load providing assemblage. As such, no moment was transferred among these structural elements. The load cell was employed to monitor the lateral loads applied to the pile head at 0.25 m above the ground surface. The head displacements of the test pile were tracked by three LVDTs attached at different heights (Figure 3b). The output signals from the strain gauges, load cell, and LVDTs were recorded and processed using a dedicated data acquisition computer.

Strain gauges were installed, in pair, on the opposite sides of the pile. As such four strain gauges wired in a full-bridge type of circuitry were used to sense the flexural behaviour at a pile depth. This arrangement allows for measurement of the pile deflection under both compression and tension and ensures the availability of output signals even when some strain gauges may 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. With this arrangement a total of 56 strain gauges were used to monitor the deflection response of the test pile under lateral loading. Two steel C shaped channels of dimensions 70x40x5 mm were attached to the opposite sides of the pile, thus covering the strain gauges and serving as protective cases during pile driving (Figure 4a). It is noteworthy that a conical protective shoe of 0.5 m long was fitted at the bottom tip of the pile so as to prevent possible damage during the pile driving as illustrated in Figure 4b.

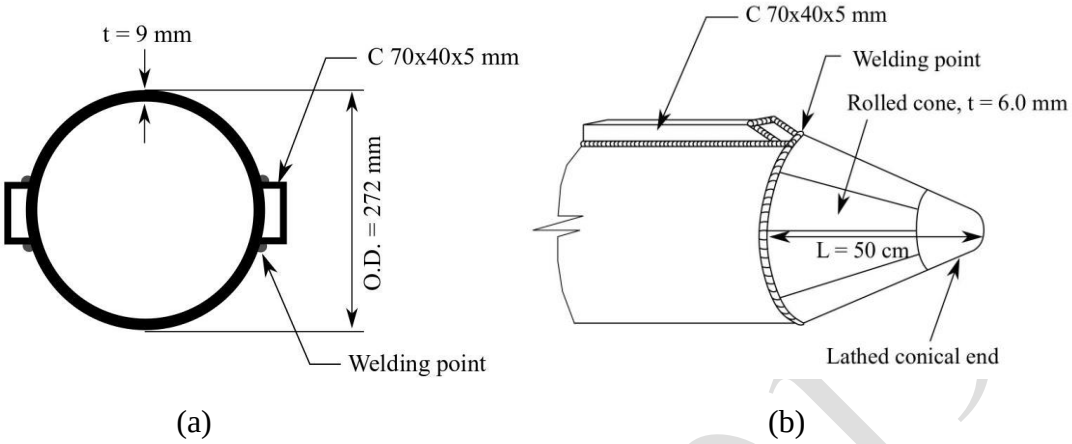


Figure 4 Details of pile cross section and conical protective shoe [17].

2.3 Testing procedure

A monotonic lateral load test was performed three months after the test pile and reaction piles were installed. This was to ensure sufficient dissipation of the pore waters in the adjacent foundation soils that were created during pile driving. Lateral loads were applied in increments of one ton until a maximum load of 11 ton was attained. In each step the hydraulic actuator was extended at rates close to 1 mm/min until the target load was achieved. The applied load was held constant for 15 minutes before proceeding to the next load step. After the peak value of 11 ton was reached, the pile was unloaded to 7, 4 and 0 ton, respectively.

2.4 Analysis of strain-gauge results.

The strain-gauge readings were analysed based on the framework of beam on elastic foundation. The pile curvature is computed by:

$$\varphi = 2\frac{\varepsilon}{D} \tag{1}$$

where ε is the longitudinal strain along the surface of the test pile and D is the pile diameter. The bending moment can be calculated as

$$M = EI\varphi \tag{2}$$

where EI is the elastic flexural stiffness of the test pile. The results of a separate four-point flexural test performed on a pile section with a length of 3 m and with a cross-section similar to that of the test pile indicates a constant value of EI of 21,000 kN-m^2 [17].

The variations of shear force (V) and soil resistance (p) 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 obtained by integrating the curvature, i.e.; $S = \int \varphi dz$ and $y = \int S dz = \iint \varphi dz dz$. In these mathematical expressions, the variable z denotes the pile depth. The differentiations of the bending moment to determine the values of V and p can be approximated using a two-point finite difference formula:

$$\frac{df}{dz} \approx \frac{f_{i-1} + f_{i+1}}{z_{i-1} + z_{i+1}} \tag{3}$$

The pile slope and deflection are determined by approximating the analytical integration using a trapezoidal rule:

$$\int f dz \approx \sum_{i=1}^{n} \left(\frac{f_i + f_{i-1}}{2}\right)(z_i - z_{i-1}) \quad (4$$

where n is the number of elevations of the installed strain gauges. When the resulting soil resistances and pile deflections are cross-plotted for all load steps, the experimental $p - y$ curves are obtained. Examples of such are presented in Figure 5 for the cases of pile depths of 1 m and 2 m.

3. Development of $p - y$ model for Bangkok clay

3.1 Review of relevant existing models for clays

Several $p - y$ models have been proposed for analysis of the load-deflection characteristics of piles in cohesive soils. Perhaps, the most notable ones are Matlock's model for soft clay [9], Reese's model for stiff clay with no free water [11], Reese's model for stiff

clay with free water [12] and the Wu $p - y$ model for medium clay [18]. These p-y models were formulated from the results of full-scale lateral pile load tests in different specific clays, and have been widely used in engineering practice owning to their simplicity and availability in commercial computer programs. The details of three $p - y$ models pertinent to the present study are provided below.

3.1.1 Matlock $p - y$ model for soft clay

The Matlock $p - y$ model for soft clay assumes a parabolic relationship between the normalized soil resistance (p/p_u) and the normalized pile deflection (y/y_{50}):

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{\frac{1}{3}} \tag{5}$$

where p_u is the ultimate soil resistance and y_{50} is a model parameter controlling the slope of the parabolic curve. The ultimate soil resistance is computed as:

$$p_u = N_p s_u D \tag{6}$$

where $N_p = [3 + \gamma' z/s_u + Jz/D]$ is a dimensionless ultimate soil resistance coefficient. The parameter γ' is the effective unit weight of soil and J is a model parameter whose value is taken as 0.25 for soft clay.

In the formulation for N_p, the first term expresses the resistance of soil at the ground surface and is derived from a rationale that the soil in front of the pile is sheared forward and upward as failure is approached. At greater depths the frontal soil is pushed forward, but forced in a manner such that the soil deformation gradually takes place more in the horizontal direction. Therefore the value of N_p increases and such is reflected in the second term that becomes larger with the effective overburden stress. The third term of N_p may be thought of as the geometrically related restraint that even a weightless soil around the pile would provide against upward flow of the soil. At great depths, the soil flows plastically and only in the horizontal direction around the pile, and N_p = 9 is recommended as an upper limit for such conditions [9,19,20,21].

The value of y_{50} is determined from the strain corresponding to 50% of the deviator stress at failure as $y_{50} = 2.5\varepsilon_{50}D$. The value of ε_{50} can be obtained from either laboratory unconfined compression (UC) test or unconsolidated-undrained triaxial (UU) test. For Matlock's model for soft clay, the value of p is determined from Equation 5 for y smaller than $8y_{50}$. The value of p is, however, limited at p_u for greater y values.

3.1.2 Reese's model for stiff clay without free water

This model for stiff clay with no free water assumes the parabolic form of normalized $p - y$ curve similar to the Matlock $p - y$ model. The exponent of the y/y_{50} term, however, is changed to 0.25. The value p_u is also determined using Equation 6, and the value of J used in determining N_p is taken as 0.5 for stiff clay. The parabolic curve is assumed to reach its maximum value of p_u and remain constant for y values equal to and greater than $16y_{50}$.

3.1.3 Wu's model for medium clay

A hyperbolic function is assumed to represent the relationship between the normalized soil resistance and pile deflection:

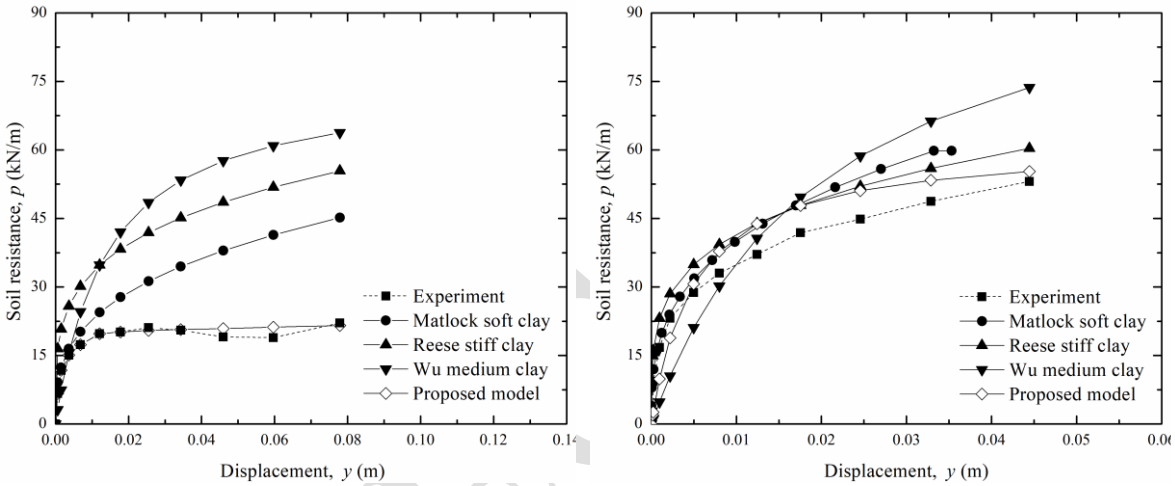
$$\frac{p}{p_u} = \frac{y/y_{50}}{2 - R_f + R_f\, y/y_{50}} \tag{7}$$

where R_f is a model parameter that defines the asymptote (p_u/R_f) of the hyperbolic $p - y$ curve. The ultimate soil resistance is defined in a manner similar to the previous models. However, the means to determine its values is simplified and is represented by a bilinear function of the pile depth. In this model the N_p value linearly increases from 2.5 at the ground surface to 10 at $z = 5D$ and remains equal to 10 for greater depths. The parameter y_{50} in Equation 7 controls the slope of the hyperbolic curve. It is computed as $y_{50} = A\varepsilon_{50}D$. The coefficient A for Wu's model is set identical to N_p as oppose to the Matlock and Reese $p - y$ models whose A values are presumed to be constant as 2.5 and 1.0, respectively.

3.2 Comparison of experimental $p - y$ curves and predictions of different models

The relevant soil properties as reported in Section 2.1 are input into the three $p - y$ models discussed in the previous section. The numerical predictions so obtained are then

plotted with the experimental $p - y$ curves in Figure 5. It can be observed that the three existing $p - y$ models for clays predict unrealistically stiff responses as compared to the experimental curves in terms of both initial stiffness and soil resistance. The key cause is likely that the models over-predict the ultimate soil resistance by a significant amount. Such high values of p_u are a result of large predicted N_p values, and this is likely due to that the failure mechanisms of the soil in front of the pile are unrealistically assumed. Based on the highlighted drawbacks, a new $p - y$ model whose predictions can better predict the experimental results is sought herewith through modification of the existing $p - y$ models.



(a) (b)

Figure 5 Experimental $p - y$ relationships obtained at pile depths of 1 m (a) and 2 m (b). Predictions of existing of *p-y* models for clays are also plotted for comparison.

3.3 $P - y$ model for Bangkok clay

Based on the shapes of the $p - y$ curves obtained from the full-scale load test, a hyperbolic function is considered most realistic to match the experimental data. The formulation of Wu's model for medium clay (Equation 7) is assumed to relate the normalized soil resistance to the normalized pile deflection:

$$\frac{p}{p_u} = \frac{y/y_{50}}{\frac{\beta}{\beta - 1} + \frac{\beta - 2}{\beta - 1} y/y_{50}} \tag{8}$$

The general shape of this hyperbolic function is shown in Figure 6. The $p - y$ curve increases and reaches the asymptote, i.e., $p/p_u = (\beta - 1)/(\beta - 2)$ at very large y/y_{50} values. However, the maximum value of p is, in reality, limited at p_u. The portion of the original hyperbolic curve above p_u is thus cut-off and is replaced by a horizontal line representing a constant value of ultimate soil resistance. In this manner the ultimate soil resistance is attained when $y/y_{50} = \beta$.

Equation 6 is also employed to compute N_p from the undrained shear strength. Proper values of N_p for this model, however, are back-calculated from the ultimate soil resistances obtained from the experimental $p - y$ curves of different depths. It has been observed that the values of N_p increase with the vertical effective stress or depth. Such a variation is simplified and assumed to be a tri-linear function of the pile depth-to-diameter ratio as shown in Figure 7. An upper limit of $N_p = 11$ is provided into the model so as to represent a bounded state of soil that undergoes a bearing-capacity type of failure at pile depths greater than $11D$.

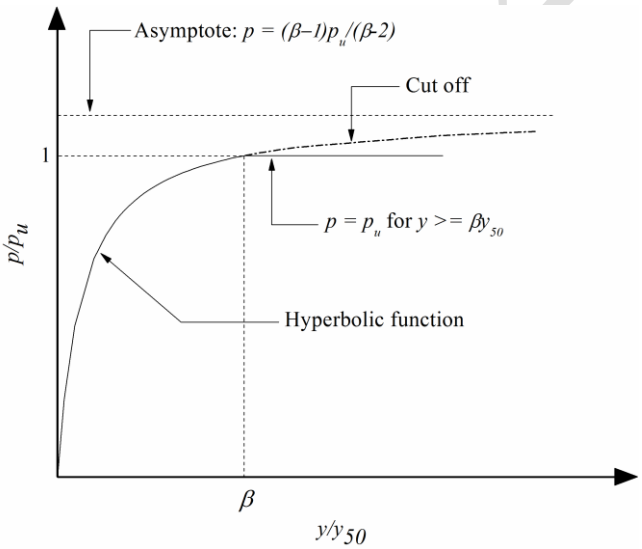


Figure 6 Hyperbolic $p - y$ model for analysis of laterally loaded piles in Bangkok clay.

The value of β can be determined from the full-scale load test results as $\beta = y_{50\,field}/y_{100\,field}$ where $y_{50\,field}$ and $y_{100\,field}$ are the pile deflections corresponding to p values 50% and 100% of the ultimate soil resistance, respectively. A value of β of 10 is found to best represent the observed pile deflections at all pile depths where the foundation soil is apparently loaded to failure and an ultimate state of p is reached and present. In the absence of full-scale test results, the value of β used to predict the pile response can be obtained as $\beta = \varepsilon_{50}/\varepsilon_{100}$ where ε_{100} is the strain corresponding to the deviator stress at

failure. Similar to Matlock's model for soft clay, the parameter y_{50} for the present model is calculated from $y_{50} = 2.5\varepsilon_{50}D$ where ε_{50} is previously defined. The values of ε_{50} and ε_{100} can be obtained from laboratory UC and UU tests as explained earlier. When laboratory stress-strain relations are not available, an ε_{50}value of 1% may be used as it has been observed to produce the most realistic $p - y$ relationships for Bangkok clay as compared to the experimental results.

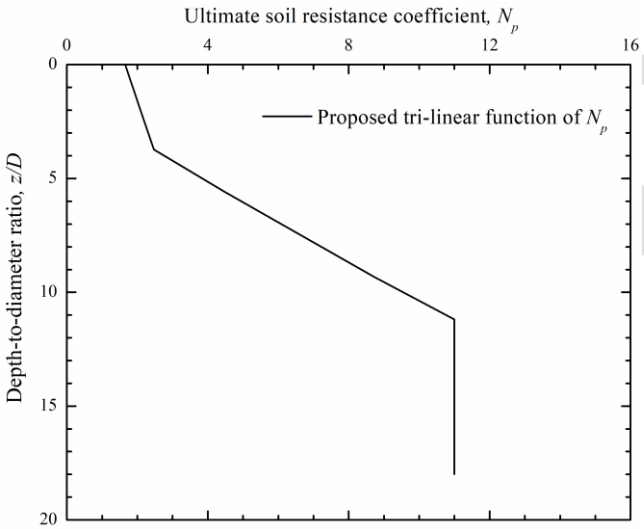


Figure 7 Variation of dimensionless ultimate soil resistance coefficient with pile depth.

The predictions of the present hyperbolic model are plotted and compared to the experimental $p - y$ curves for different depths as shown in Figure 8. It can be observed that the numerical predictions can match the test results with good accuracies both in terms of initial stiffness and ultimate soil resistance, particularly for small pile depths. Some discrepancies between the predicted and experimental curves can be observed for $z \geq 2$ m. For such cases, the numerical curves increase more rapidly for small y values, but only marginally afterwards. Considering the trend of these curves, it appears that the predicted $p - y$ relationships would reach peak values that are smaller than the probable experimental ultimate soil resistances that would otherwise take place at greater pile deflections.

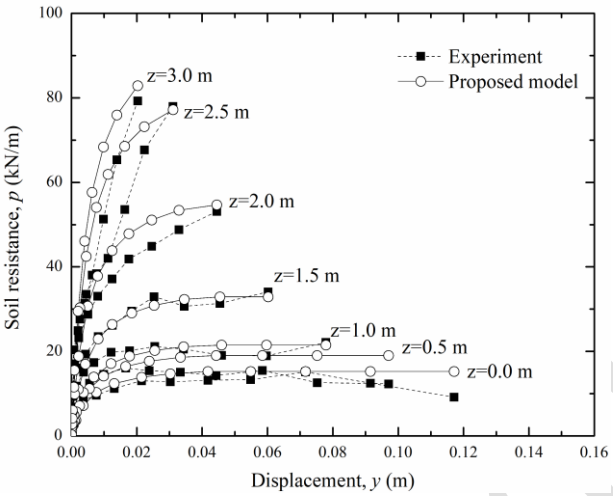


Figure 8 Predictions of hyperbolic $p - y$ model for Bangkok clay as compared to experimental data

4. Simulations of load-displacement behaviour

The hyperbolic $p - y$ model for Bangkok clay is implemented into the commercial software LPILE so as to perform numerical simulations of the full-scale lateral load test. The pile properties, details of soil profile and values of relevant parameters such as shear strength and unit weight are entered into the software and a load-displacement control analysis is performed with a free-head type of fixity. The magnitude of lateral load P applied at the pile head is increased stepwise and the corresponding soil resistances, shear forces, bending moments, slopes and deflections are computed. The simulated load-displacement relations at the pile head and the variations of bending moment, deflection and soil resistances with depth for load steps of 5 and 11 ton are shown in Figures. 10, 11 and 12, respectively. In these figures, the experimental results are also plotted for comparison purpose.

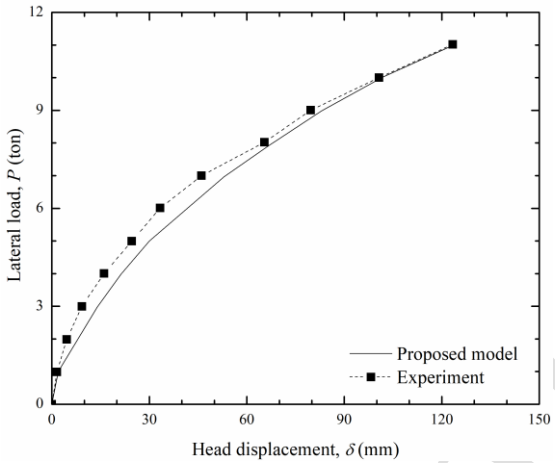
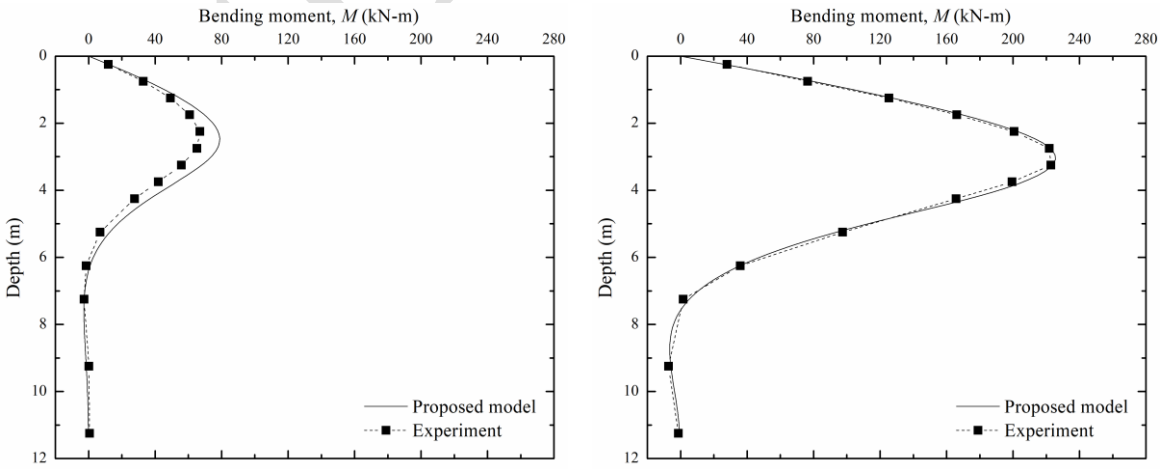


Figure 9 Predicted and experimental load- head displacement relationships.

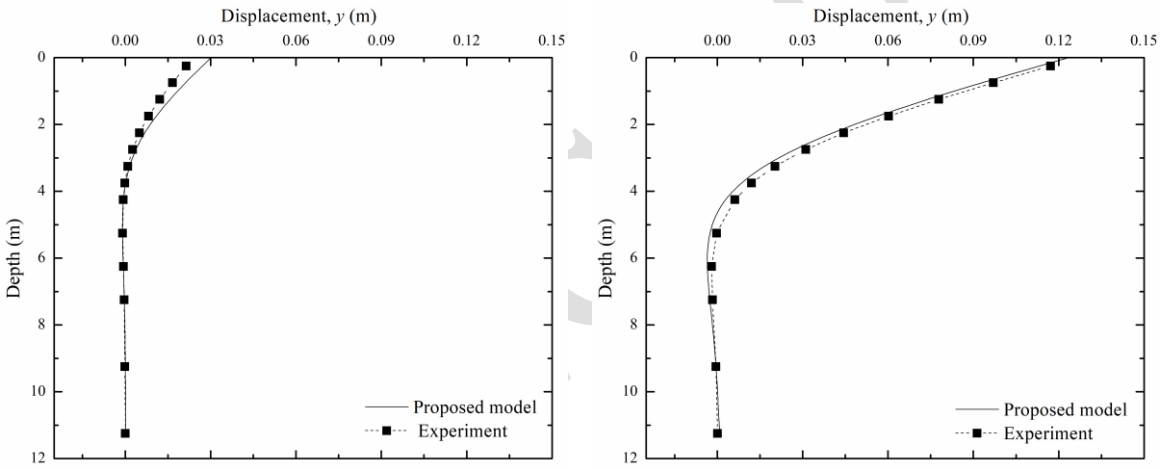
The experimental $P-\delta$ curve in Figure 9 suggests a stiff load-displacement response for small displacements. For large displacements, the curve continues to gradually increase, however, at smaller rates. The lateral load test was discontinued at 11 ton, and a maximum head displacement of 125 mm was observed in response. The numerical predictions of LPILE in conjunction with the proposed $p-y$ model marginally over-predict the head displacements particularly for intermediate lateral loads. As P increases, such a deviation gradually decreases, and the predicted displacements become practically identical to the experimental values for lateral loads greater than 8 ton.



(a) (b)

Figure 10 Variations of pile bending moment with depth for load steps of (a) 5 ton and (b) 11 ton.

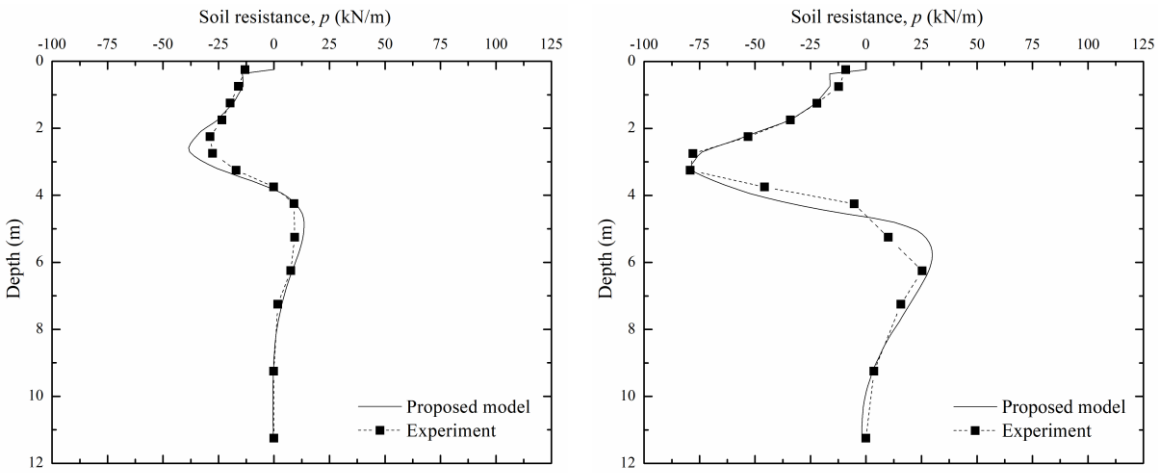
It can be observed in Figure 10 that the maximum bending moments in the test pile increase and take place at slightly greater depths with increasing P values. The locations of maximum bending moments are within 3.5 m or 13B below the ground surface. Such an extent is greater than the upper limit of 8-10B usually reported in the literature [22]. The $p-y$ model predicts slightly larger bending moments as compared to the experimental values for P = 5 ton. Excellent agreement between the model predictions and the experimental bending moments, however, can be observed for the case of P = 11 ton.



(a) (b)

Figure 11 Experimental pile deflections compared with LPILE predictions for (a) 5-ton and (b) 11-ton load steps.

As shown in Figure 11a, the pile deflections are also slightly over-predicted for an intermediate P value of 5 ton. These larger predicted deflections, particularly for z < 12D, are likely the key cause for the bending moments being over-predicted for the same load step as reported earlier. For the maximum load step of 11 ton, the proposed $p-y$ model can simulate the experimental results with excellent accuracies (Figure 11b). It can be observed from Figure 12 that the numerical predictions of soil resistance agree well with the experimental results for both 5-ton and 11-ton load steps.



(a) (b)

Figure 12 Comparison of numerical and experimental soil resistances for load steps of (a) 5 ton and (b) 11 ton.

4 Conclusions

The main purpose of this research is to develop a realistic *p-y* model for analysis and design of laterally loaded piles in Bangkok clay. The results of a full-scale lateral load test served as a basis in such a development. A hyperbolic correlation was proposed to represent the normalized soil resistance-deflection relationships. To determine the ultimate soil resistance for each pile depth from undrained shear strength, a new trilinear function was introduced so as to compute for N_p value. The present $p - y$ model was able to predict the experimental curves with good accuracies. When it was implemented into the finite-difference based software LPILE, the hyperbolic $p - y$ model could, in general, simulate the experimental load-head displacement relations and the variations of bending moment, soil resistance and pile deflection with depth with excellent accuracies. This suggests a very high level of simulative capability of the model and its potential in practical use for analysis and design of deep foundations under lateral loading in Bangkok clay.

Acknowledgement

The full-scale experiment presented herein was funded by the Thailand Research Fund (TRF) and Seafco Public Company Limited through Grant MSD5710050. The authors would like to thank Mr. Gong Chaosittichai for his assistance in analysing and preparing the experimental results used in the development and validation of the proposed model.

References

[1] Coduto, D. P. (1994). **Foundation Design; Principles and Practices**. Englewood Cliffs, New Jersey: Prentice Hall.

[2] Reese, L. C., Isenhower, W. M., & Wang, S. T. (2006). **Analysis and Design of Shallow and Deep Foundations.** New Jersey. John Wiley.

[3] Salgado, R. (2008). **The Engineering of Foundations.** New York: McGraw-Hill.

[4] Poulos, H. G., & Davis, E. H. (1980). **Pile Foundation Analysis and Design**. Taipei: Rainbrow bridge book Co.

[5] Broms, B. B. (1965). "Design of laterally loaded piles." **Journal of Soil Mechanics & Foundations Division ASCE**. Vol.91. Issue 3 : 79-99.

[6] Hetenyi, M. (1946). **Beams on Elastic Foundations**. Scientific series. Ann Arbor : University of Michigan Press.

[7] Matlock, H., & Reese, L. C. (1962). "Generalized solutions for laterally load piles." **Transactions of the American Society of Civil Engineers**, Vol.127. Issue 1 : 1220-1247.

[8] Isenhower, W. M., Wang, S. T.,& Vasquez L. G. (2016). **Technical Manual for LPILE 2016**. Austin, Texas : ENSOFT. Inc.

[9] Matlock, H. (1970). **"**Correlations for design of laterally loaded piles in soft clay.**"** **Proceeding of the 2nd Annual Offshore Technology Conference, Houston, Texas** : 577-594.

[10] Reese, L. C., Cox, W. R., & Koop, F. D. (1974). "Analysis of laterally loaded piles in sand." **Offshore Technology in Civil Engineering Hall of Fame Papers from the Early Years**. Dallas, Texas : 95-105.

[11] Reese, L. C., & Welch, R. C. (1975). "Lateral loading of deep foundations in stiff clay". **Journal of Geotechnical and Geoenvironmental Engineering**. Vol.101. Issue GT7 : 633-649.

[12] Reese, L. C., Cox, W. R., & Koop, F. D. (1975). "Field testing and analysis of laterally loaded piles in stiff clay**.**" **In Proceedings of Seventh Annual Offshore Technology Conference II, Houston, Texas** : 671-690.

[13] Chaosittichai, G., & Anantanasakul, P**. (**2018a**) "**Full-scale lateral load tests of driven piles in Bangkok clay." ***Proceedings of the International Foundations Congress and Equipment Expo 2018***. 5-10 March 2018. Orlando, Florida : 321-330.

[14] Chaosittichai, G., & Anantanasakul, P. (2018b). "Field experimental study of load-deflection behavior of driven piles in soft Bangkok clay". **Kasem Bundit Engineering Journal**, Vol.8 : 373-385.

[15] Chaosittichai, G., & Anantanasakul, P. (2018c). Full-scale lateral load tests to determine load-displacement characteristics of driven piles in soft clay. ***Proceedings of the 5th GeoChina International Conference.*** 23-25 July, 2018. Hangzhau, China : 125-135.

[16] Horpibulsuk, S., Shibuya, S., Fuenkajorn, K., & Katkan, W. (2007). "Assessment of engineering properties of Bangkok clay." **Canadian Geotechnical Journal**, Vol.44. Issue 2 : 173-187.

[17] Chaosittichai, G. (2018). **Development of Suitable *P-y* Relationships for Analysis of Load-Displacement Characteristics and Design of Laterally Loaded Driven Piles in Sub-Soils of Greater Bangkok Area**. Master's thesis. Faculty of Engineering. Mahidol University.

[18] Wu, D., Broms, B. B., & Choa, V. (1998). Design of laterally loaded piles in cohesive soils using p-y curves. **Soils and Foundations**. Vol.38. Issue 2 : 17-26.

[19] McClelland, B., & Focht, J. A. (1958). "Soil modulus for laterally loaded piles." **Journal of Soil Mechanics and Foundations Division ASCE.** Vol.82. Issue 4 : 1-22.

[20] Skempton, A. W. (1951). "The bearing capacity of clays." **In Proceedings of Building Research Congress**. London Vol.1 : 180-189.

[21] Tschebotarioff, G. (1973). **Foundations, Retaining and Earth Structures-The Art of Design and Construction and its Scientific Bases in Soil Mechanics.** Edition 2. New York : McGraw-Hill.

[22] Duncan, J. M., Evans Jr, L. T., & Ooi, P. S. (1994). "Lateral load analysis of single piles and drilled shafts." **Journal of Geotechnical Engineering**, Vol.120. Issue 6 : 1018-1033.

Authors' profiles

Bhaskhar Pathak, B. Eng. Graduate Assistant, Department of Civil and Environmental Engineering, Faculty of Engineering, Mahidol University, Nakhon Pathom, 73710 Thailand. Email: bhashkar.pathak11@gmail.com, Tel +660802161046

Pongpipat Anantanasakul, Ph. D. Assistant Professor, Department of Civil and Environmental Engineering, Faculty of Engineering, Mahidol University, Nakhon Pathom, 73710 Thailand. Email: pongpipat.ana@mahidol.ac.th, Tel +66-81-909-9421

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