XL CILAMCE
IBERO-LATIN AMERICAN CONGRESS ON COMPUTATIONAL METHODS IN ENGINEERING
NOVEMBER 11-14, 2019
Praiamar Natal Hotel & Convention
Natal, RN-BRAZIL

COMPARATIVE STUDY OF SOIL-STRUCTURE INTERACTION METHODS FOR THE ANALYSIS OF LATERALLY LOADED PILE FOUNDATIONS

Vinícius M. Spricigo
Maria L. R. Mineiro
Ludmily S. Pereira
Eduardo M. B. Campello
Pedro W. G. N. Teixeira
vinicius.spricigo@usp.br
marialuisamineiro@usp.br
ludmily.pereira@usp.br
campello@usp.br
pedro-wellington@uol.com.br
University of São Paulo
Av. Prof. Almeida Prado, Trav. do Biênio, 83, Cidade Universitária, 05508-900, São Paulo-SP, Brazil

Abstract. When foundations are subjected to significant horizontal loads, as in bridges or offshore structures, a more in-depth research of the soil-structure interaction is of utmost importance. Different methods are given to carry out such study, among which the Winkler spring models and continuum models stand out. This work addresses these methods for the analysis of laterally loaded piles inserted in clayey and sandy soil. For the discrete spring models, both linear springs with a plasticity criterion and nonlinear springs defined by given p-y curves are adopted. For the continuum model, the soil plasticity is described by a Mohr-Coulomb criterion within a finite element method (FEM) approach. As results, the horizontal displacements of the pile-soil sets are taken and a constant of horizontal subgrade reaction is estimated, which are compared to results from load tests performed in Brazilian soils of Campinas-SP and Ilha Solteira-SP. In general, displacements obtained by the analysis methods were higher than the load tests in the initial elastic part and became smaller at a final yielding phase. The analysis points to the importance of considering concrete as an elastoplastic material and of precisely defining the Young's modulus of concrete and the initial stiffness of the superficial soil. Among the analysis methods, the linear spring model stands out for its satisfactory results and simplicity, in terms of appliance and input data. On the other hand, the nonlinear spring and continuum methods require specific geotechnical tools, but already contemplate iterative load application, which facilitates the account of concrete's nonlinearity.

Keywords: pile foundations, soil-structure interaction, nonlinear analysis, p-y curves.

1 Introduction

The verification of a building's behavior towards the soil massif that supports it is one of the main challenges faced in structural design, which is called the analysis of soil-structure interaction (SSI). Over time, a series of methodologies have been developed regarding different scenarios of analysis, being the possible divisions of the structural system an important point of discussion.

In structural design offices, the adoption of a structural system model that separates the foundation from the superstructure, which is bounded in the base by fixed or pinned supports, is a common practice. In this case, the efforts taken in the base elements (axial forces, shear forces and bending moments) are applied as loads in a foundation model and then transferred to the ground. This simplified analysis is mainly used due to the difficulty of calculation that involves the interdependence of foundation dimensioning and superstructure efforts. Another important factor is related to the problematic prediction of soil behavior and its non-linear response owing to the heterogeneity of the material, the significant modification in time and the lack of sufficiently reliable information about it [1].

Choosing a suitable foundation model is also a difficult task as the system's behavior can vary greatly depending on the soils and foundations types or applied loading. A specific case of study, frequently researched, is the one of deep foundations subjected to significant horizontal loading. According to De Beer [2], these laterally loaded piles can be classified into two groups: active piles, which transmit horizontal forces to the ground, under external loading at the top; or passive piles, when horizontal efforts along the pile are produced due to the movement of the surrounding soil. They may also be classified as long or short piles, considering its relative length in comparison to the total stiffness of the pile-soil system [3].

In literature, a first attempt of simulating the behavior of the soil on the structure were analytical evaluation models, based on a) tension, b) displacements and strains, c) the elastic problem and d) the plastic problem [4]. Since the use of analytical expressions can be a labored and time-consuming matter, these methods have little application in the routines of calculation offices. Alternatively, discrete models were proposed for deep foundations, in which the soil can be represented by springs or as a continuum medium.

The Winkler models [5] take into account the soil as an elastic medium, characterized by a series of discrete springs along the foundation domain. In these models, the coefficient of horizontal subgrade reaction, which associates the applied pressure with displacement in the same direction, has the same physical meaning of the springs' stiffness coefficients [4]. Seeing that the simulated elastic behavior is distant from reality when higher loads are applied, adapted methods grounded on the Winkler hypothesis are often used when studying piles subjected to horizontal loads, considering linear responses with a plasticity criterion or non-linear responses. In the first case, the springs present an elastic behavior until they reach stress limit conditions, when they begin to deform without offering greater resistance. In the second case, the stiffness coefficients are usually obtained from load-deformation curves (p-y curves), which can be characterized using the formulation based on experimental tests such as the one provided by the American Petroleum Institute (API) [6].

This model is commonly used in structural designs [7] and it is subject of research on SSI in piles worldwide, as can be seen in the studies of Anoyatis & Lemnitzer [8], Zhang, Deng & Ke [9] and Hassan [10]. Figure 1 illustrates a schematic example of the soil-structure interaction modeling in a single pile by a non-linear spring method, with different profile responses along the soil layers.

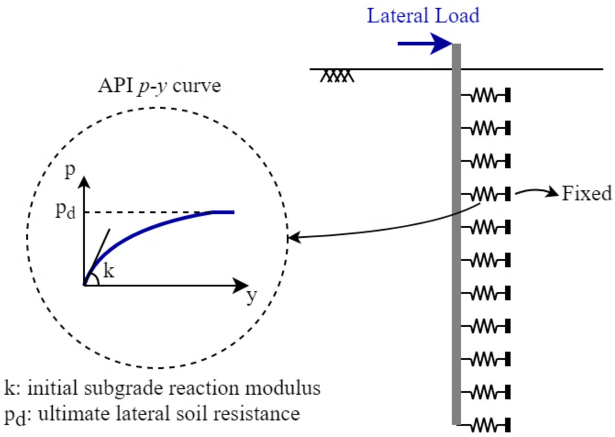


Figure 1. *p-y curves* spring method for laterally loaded pile (adapted from Rahmani et al. [11]).

In the continuum model, on the other hand, the soil can be analyzed considering its elastic or elastoplastic behavior, which requires a numerical solution using, for example, Finite Elements Method (MEF). With the advancement in storage capacity and processing speed of computers, this is a currently accessible method for designers to analyze very complex problems quickly and easily, considering two-dimensional or three-dimensional finite elements [12].

The elastoplastic analysis becomes more important depending on the degree of the foundation deformability or the acting lateral loads relevance. In the case of pile-supported bridges, offshore structures and telecommunication towers, for example, horizontal stresses such as wind, braking, centrifugal acceleration and ground vibration are significant [13]. The structure locking usually does not work as in conventional buildings and, consequently, the bending moments and lateral loads must be integrally resisted by the foundation elements. In fact, ABNT NBR 6122/2010 [14] states that when piles are subjected to horizontal forces or moments, the yielding of the soil or structural element may occur, which must be then taken into account in design with the respective deformations.

This paper aims to examine three different approaches of SSI analysis for laterally loaded concrete piles: linear spring model, nonlinear spring model and continuum model. For this, lateral load tests taken in typical soils of São Paulo/Brazil are studied considering all these types of soil representation, and the advantages and drawbacks of each method are thus discussed.

2 Methodology

The static lateral load tests studied were carried out by Miranda [15] and Del Pino Jr. [16], in experimental fields from State University of Campinas (UNICAMP) and São Paulo State University (UNESP) located in the Brazilian cities of Campinas-SP and Ilha Solteira-SP, respectively. Figure 2 illustrates the schematic configuration of both tests considering a single excavated concrete pile with free head. The informed diameter d and Young's Modulus E of both piles are also presented.

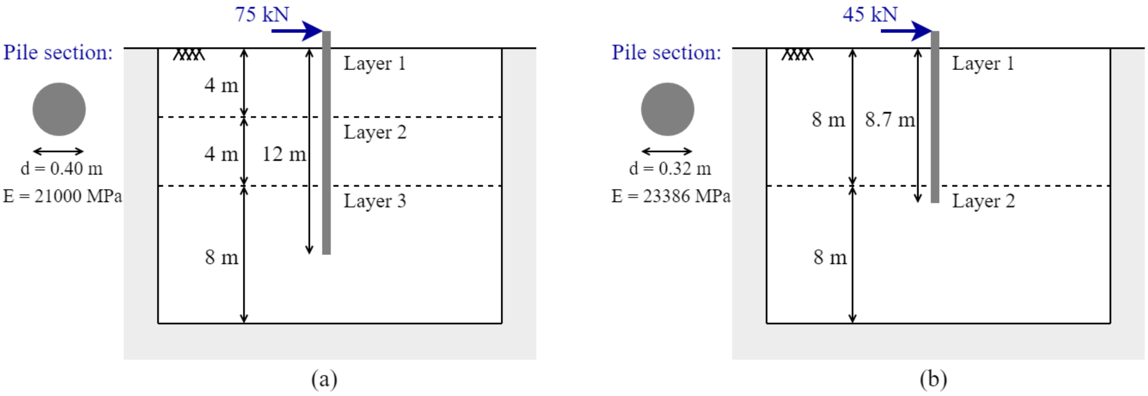


Figure 2. Lateral load tests scheme in (a) Campinas-SP and (b) Ilha Solteira-SP.

The soil of the load tests set for 12 meters piles, in the northern region of Campinas-SP, consists basically of diabase residual soil (clays and silts) with low values of penetration resistance, and the water level is below 17 m deep [17]. Table 1 shows the average geotechnical properties of the soil admitted, per layer: soil type, specific weight (γ), friction angle (φ), cohesion (c) and the layer's average penetration resistance index (N_{SPT}) taken from Standard Penetration Tests (SPT). The secant Young's modulus for the soil E_s admitted, defined by empirical correlations with the N_{SPT} values [18], and the Poisson's coefficient ν, based on Bowle's studies [19], are also presented.

Table 1. Characteristics of Campinas-SP soil (adapted from data provided by Albuquerque [17])

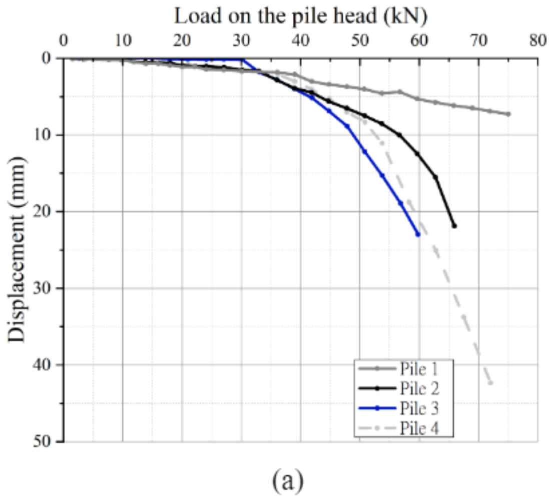
Layer	Type	Depth (m)	γ (kN/m³)	φ (°)	c (kPa)	N_{SPT}	E_s (kPa)	ν
1	Sandy-silty clay	0-4	13.0	30	5	3	6300	0.30
2	Silty-sandy clay	4-8	15.0	21	22	6	13860	0.30
3	Clayey-sandy silt	8-16	16.0	19	72	8	10000	0.30

The soil of the experimental field located in Ilha Solteira-SP, where the second group of load tests was performed, is composed of a first colluvial sandy soil layer of approximately 11 meters followed by an alluvial soil layer [16]. Its assumed properties, in average per layer, are summarized in Tab. 2.

Table 2. Characteristics of Ilha Solteira-SP soil (adapted from data provided by Del Pino Jr. [16])

Layer	Type	Depth (m)	γ (kN/m³)	φ (°)	c (kPa)	N_{SPT}	E_s (kPa)	ν
1	Clayey sand	0-8	15.0	33	3	4	7200	0.30
2	Clayey sand	8-16	18.0	30	18	8	14400	0.30

Considering the piles dimensions, loads and the soils properties, a computational code was developed based on the Winkler hypothesis with a linear or non-linear approach in order to calculate the horizontal displacements of the piles and the supporting reactions and bending moments. The analysis was also made assuming a continuum medium, in a geotechnical software that uses FEM for evaluation of soil behavior. Both piles are defined as active and long and, therefore, were treated as flexible beams. The results regarding the soil deformation and rupture were then compared with the data previously obtained by Miranda [15] and Del Pino Jr. [16] from the lateral load tests, presented in Figures 3a and 3b.



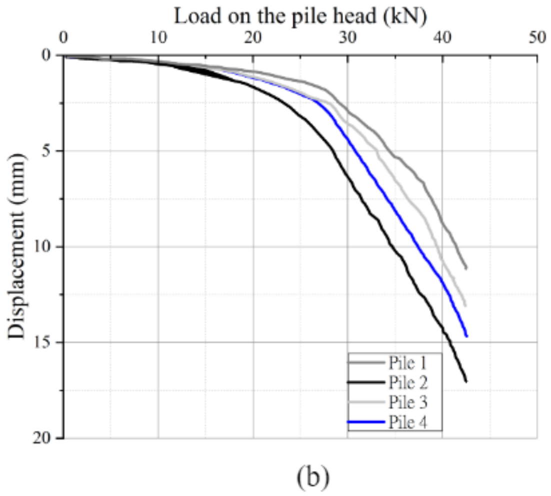


Figure 3. Lateral load test results for excavated piles in natural soil, in (a) Campinas-SP and (b) Ilha Solteira-SP.

Additionally, to better evaluate the suitability of the analysis methods for the elastic phase of the soil's response, when pile loads and displacements are lower, the equivalent parameters of the constant of horizontal subgrade reaction (η_h) were calculated for each method. These parameters, according to Terzaghi's [20] classical work, can represent the linear growth of the soil's coefficient of horizontal subgrade reaction with increasing depth (K).

This same calculation is made by Miranda [15] and Del Pino [16] to designate the physical behavior of the soil investigated in elastic regime, by taking the corresponding forces (H) in the load test at an interval of top displacement (y_0y0) between 6 mm and 12 mm, and through the interpolation of values obtained for η_h. According to the theory of Matlock and Reese [3], these are given by the following equation:

$$\eta_h = \frac{4{,}42\, H^{5/3}}{{y_0}^{5/3} (EI)^{2/3}} \tag{1}$$

Based on the work of Terzaghi [20], Matlock and Reese [3] concluded that the behavior of the system is mainly defined by the soil that occurs within the depth T, which represents the relative stiffness between pile and soil. For the pile to be classified as long – and for the methods explored in this paper to be valid – the length of the pile must be at least 4 times the value of T, which can be calculated as presented in Eq. (2) [3].:

$$T = \sqrt[5]{\frac{EI}{\eta_h}} \tag{2}$$

2.1 Linear Spring Model with Plasticity Criterion

A program was developed in MATLAB to analyze the horizontal deformation of the soil adjacent to the long piles studied. Firstly, a linear analysis was admitted for the pile and for the soil. Each pile was defined as a beam according to the Bernoulli-Euler theory, divided into n elements of the same length, and the soil was represented by discrete springs, in each 20 cm, based on the Winkler model. The routine involves the global stiffness matrix determination and the calculation of the horizontal displacements and springs support reactions. In sequence, a plasticity criterion was adopted to consider the material non-linearity of the soil, correspondent to an iterative process of comparison between the reactions of the springs and the maximum reactions when there is plastification of the soil.

In the case of a pile subjected to lateral load, the spring stiffness coefficients can be obtained through the horizontal reaction coefficient variable with depth. The formulation for long piles is presented in Tab. 3, being K a coefficient dependent of the soil type considering Aoki & Veloso's classification [21] and Δz the height adopted for each beam element. For sandy-silty clay, silty-sandy clay, clayey-sandy silt and clayey sand, the values of K are, respectively, 30, 33, 25 and 60 tf/m².

Table 3. Spring stiffness coefficient.

Variable	Equation	Studies	
Base resistance	$q_c = K \cdot N_{SPT}$	Aoki & Velloso [21]	(3)
Coefficient of horizontal subgrade reaction	$k_h = 4.5 \cdot \frac{q_c}{d}$	Marche [22]	(4)
Spring stiffness coefficient	$k_m = \Delta z \cdot d \cdot k_h$	(Area of influence)	(5)

Then, the global stiffness matrix $\mathbf{K}$ was composed by the stiffness matrix of each bar element $\mathbf{K_e}$ and the spring stiffness coefficient k_m at the corresponding degrees of freedom. The lateral load was assigned to create the load vector $\mathbf{F_u}$. The nodal horizontal displacements $\mathbf{U_u}$ were obtained from the solution of the equations system according to the matrix structural analysis (Eq. 6):

$$\boldsymbol{KU} = \boldsymbol{F} \tag{6}$$

As the regime considered is purely elastic, the horizontal displacements of both pile and soil are the same. The spring reactions were calculated by multiplying the spring stiffness coefficients by the displacements provided. To determine the limiting capacity of the springs, the maximum spring reactions R_{max} were calculated, by equilibrium, as a function of passive and active earth pressure, as shown on Tab. 4, in which z is the height of the evaluated point and the considered load spreading factor is equivalent to 3 for passive earth pressure.

Table 4. R_{max} calculation.

Variable	Equation	Studies	
Coefficient of active earth pressure	$k_a = \dfrac{(1 - sin\,\varphi)}{(1 + sin\,\varphi)}$	Rankine [23]	(7)
Coefficient of passive earth pressure	$k_p = \dfrac{(1 + sin\,\varphi)}{(1 - sin\,\varphi)}$		(8)
Active earth pressure	$\sigma_a = k_a \cdot \gamma \cdot z \; - 2c\sqrt{k_a}$	Bell [24]	(9)
Passive earth pressure	$\sigma_p = k_p \cdot \gamma \cdot z + 2c\sqrt{k_p}$		(10)
Resultant active earth pressure	$E_a = \Delta z \cdot d \cdot \sigma_a$	(Equilibrium)	(11)
Resultant passive earth pressure	$E_p = 3\Delta z \cdot d \cdot \sigma_p$		(12)
Maximum reaction	$R_{max} = E_p \; - \; E_a$		(13)

When the reaction of a spring exceeded R_{max}, the spring lost its load capacity and it was replaced by a force of intensity R_{max}, being the nodal displacements then recalculated. The process was performed iteratively until the new spring reactions did not exceed the limits allowed by the plasticity criterion. The outputs data of the program were the number of times the soil-structure system reached the plasticity criterion and the horizontal nodal displacement *vs* depth graph.

2.2 Nonlinear Spring Model

The nonlinear discrete models of the examined piles were developed with the aid of RSPile v1.0, a general pile analysis software for analyzing laterally loaded piles [25]. The nonlinear spring analysis allows the observation of the foundation behavior under greater loads and displacements, when the most superficial portion of soil assumes a highly nonlinear behavior and may reach the ultimate soil reaction. The springs are well represented by typical curves of lateral pressure (p) and displacement (y), called p-y curves. Since each type of soil has its own resistance and deformability characteristics, several models have been proposed to generate these curves. Two of the most commonly used derive from studies performed by Matlock (1970) and Reese et al. (1970), which are applied for soft clays with free water and sands, respectively [26]. These were the approaches considered in this paper.

The equations that define Matlock's model for soft clays with free water are highlighted in Tab. 5. The p-y curves are simply defined by a nonlinear curve that reaches a top level of constant pressure (p_{ult}). The required input data are the effective unit weight (γ'), the undrained shear strength (c_u), the strain corresponding to one-half the maximum principal stress difference (ε_{50}) and a material related parameter (J). According to Peck, Hanson & Thorburn [27], the default values of ε_{50} and J for soft clays are 0,020 and 0,50, respectively.

Table 5. Equations of p-y curves for soft clays in the presence of free water (Matlock (1970), apud Reese & Van Impe [25])

Variable	Equation	
Curve equation	$p/p_{ult} = 0{,}5 \cdot (y/y_{50})^{1/3}$	(14)
Ultimate soil reaction	$p_{ult} = máx\begin{cases}[3 + (\gamma'/c)\,.\,z + (J/d) \cdot z] \cdot c_u \cdot d \\ 9 \cdot c_u \cdot d\end{cases}$	(15)

The method of Reese et al. (1974) for sands above and below free water is described by the formulation presented in Tab. 6. In this case, the p-y curves have 4 different stages: it starts in a linear path with stiffness k_{py}, then assumes a nonlinear curve at pressure p_k, becomes linear again at p_m (y_m = b/60) and finally reaches a runoff level with constant pressure at p_u (y_u = 3b/80). Additional entry data for the model are the soil the friction angle φ, the active earth pressure coefficient k_a, the resting earth pressure coefficient k_0 and the soil unit weight γ – buoyant or total, depending on the water level. A typical value of k_{py} for loose sand above the water is 6,8 MN/m³ [25]. The parameters $\overline{A}_s$ and B_s vary along the depth and are taken from the abacus provided by Reese et al., and k_0 can be assumed as 0,40.

Table 6. Equations of p-y curves for sands above and below free water (Reese et al. (1974), apud Reese & Van Impe [25])

Variable	Equation	
Coefficients	$\alpha = \varphi/2;\ \beta = 45 + \varphi/2$	(16)
Pressure Limits	$p_s = mín\begin{cases}\gamma.z.\left[\dfrac{k_0.z.tan\,\varphi\,.sin\,\beta}{tan(\beta-\varphi)\,.cos\,\alpha}) + \dfrac{tan\,\beta}{tan(\beta-\varphi)}(d + z.tan\,\beta\,.tan\,\alpha) \right. \\ \left. +k_0.z.tan\,\beta\,.(tan\,\varphi\,.sin\,\beta - tan\,\alpha) - k_a.d\right] \\ k_a.d.\gamma.z.(tan^8\beta - 1) + k_0.d.\gamma.z.tan\,\varphi\,.tan^4\beta\end{cases}$ $p_u = \overline{A}_s \cdot p_s;\ p_m = B_s \cdot p_s$	(17)

The soil of Campinas-SP was simulated by both soft clay with free water and sand p-y curves, since it has a predominance of clay in the first layers (mean value of 55,5 %), but also a high content of sand and silt (27,0 % and 17,5 %, respectively), which are better represented by sand models. The soil of Ilha Solteira-SP was simulated only by p-y curves for sands.

The software RSPile v1.0 was employed to generate such curves for the cases studied [25]. It makes an iterative calculation, with a default of 100 steps and convergence tolerance of $1{,}00.10^{-4}$, using the Bernoulli-Euler beam theory to calculate the pile displacements for each loading step. For the analysis, the piles were divided into 20 cm segments.

2.3 Continuum Model

The two-dimensional continuum models of the lateral load tests were elaborated using the software *Phase²* 7.0, by Rocscience, a commercial finite element software for geotechnical analysis of soils deformations and stresses around underground excavations. Closed polylines represented excavations and the soil mass was automatically discretized in a mesh of 6 nodes triangular elements, which was refined in the contact region between the pile and the soil, since the mesh type chosen was graded. For the boundaries, it was assumed that the soil mass total dimension was 40 m in length by 20 m in height, and its horizontal displacements in the lateral contours and horizontal and vertical displacements in the bottom were restricted. The model developed for the pile in Campinas-SP is illustrated in Fig. 4.

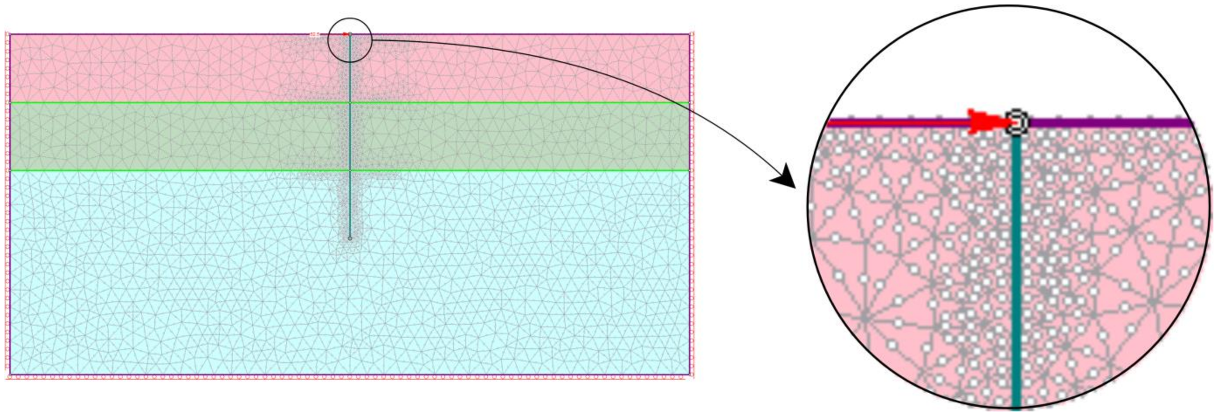


Figure 4. 2D Continuum model mesh used in *Phase²*

When assuming a 2D model in Plane Strain, the software considers that the massif has infinite length normal to the plane section of the analysis, so that all the input data and results are adopted per meter [28]. Then, the major and minor in-plane principal stresses, the out-of-plane principal stress and the in-plane displacements and strains are calculated. To overcome the fact that the piles are simulated as a bar element, but they should represent a continuous concrete curtains, it is possible to adopt the same concept of spreading factor equal to 3 as in the springs models. This way, it was assumed in the analysis that an effort applied in the 40 cm diameter pile (Campinas-SP) should be resisted by a soil slice of 120 cm, while an effort in the 32 cm diameter pile (Ilha Solteira-SP) should require a slice of approximately 100 cm soil slice, that is, a 100 cm thickness. The others parameters, such as the applied loading, were also adapted to fit the model dimensions.

In the model, the soil was defined as a plastic material, and the failure criteria selected for describing its strength was Mohr-Coulomb, which enables the occurrence of plastic points in the model. With this consideration, in order to avoid yielding, the principal stresses observed at any point in the soil cannot exceed a surface defined as a function of the parameters of friction angle φ, cohesion c and expansion angle ψ (usually null) [29]. For the interaction between structure and soil, the same Mohr-Coulomb criterion was adopted, but the considered friction angle was multiplied by a factor of 2/3 [1]. Besides these variables, the specific weight γ, the secant Young's modulus of the soil E_s and the Poisson's coefficient ν were also input data of the program.

Although the relationship between tension and deformation in soils generally presents non-linear behavior, in the elastic-linear model of Mohr-Coulomb, *Phase²* only calculates the elastic stiffness matrix of the soil at the beginning of the process. Then, it starts an iterative process of increasing stresses, updating the directions and magnitudes of the loads and deformations at each cycle without changing the stiffness matrix for the undisturbed condition. This model loses precision, therefore, as the deformations imposed on the soil increase.

3 Results and Discussions

3.1 Graphics of Maximum Pile Deflection

The first results, shown in Figures 5 to 8, are the graphics of maximum horizontal deflections of the soil-pile set for each model. These graphics refer to the final loads applied in each case, which correspond to 75 kN for the soil in Campinas-SP and 45 kN for Ilha Solteira-SP. Figure 5 illustrates the results for pile deflection along the soil depth considering the linear spring model adopted.

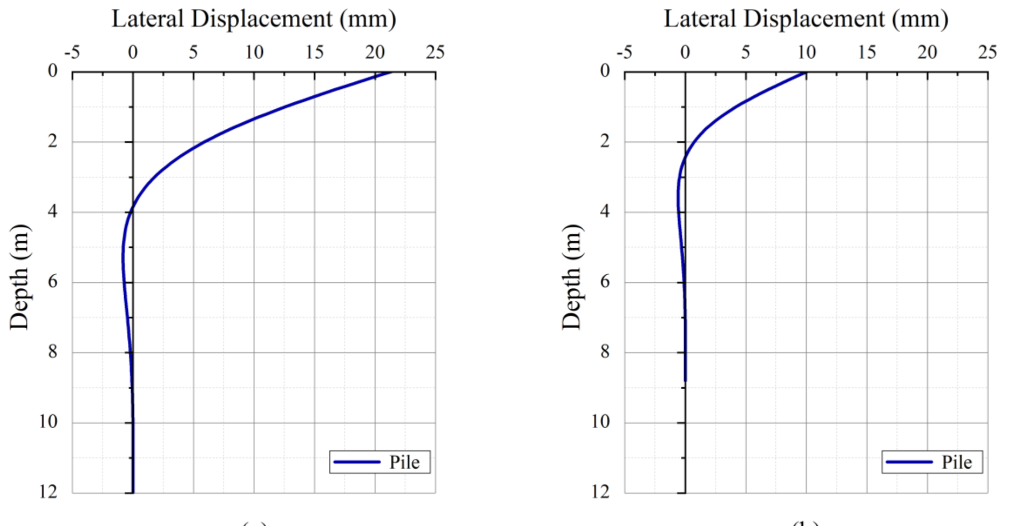


Figure 5. Maximum pile deflection for (a) Campinas-SP and (b) Ilha Solteira, determined by the linear springs with plasticity criterion model (mm).

The lateral displacements provided by the nonlinear spring model are presented in Fig. 6 for both piles. The analysis of the excavated pile in Campinas was performed considering both clay p-y curves and sand p-y curves. For the pile in Campinas-SP, it is observed that the maximum displacements when clay p-y curves are applied are almost 8 times higher than the ones with sand p-y curves. In both sand models, the displacements become close to zero at a 3 meters depth, while in the clay model for Campinas this depth is approximately 6,5 meters.

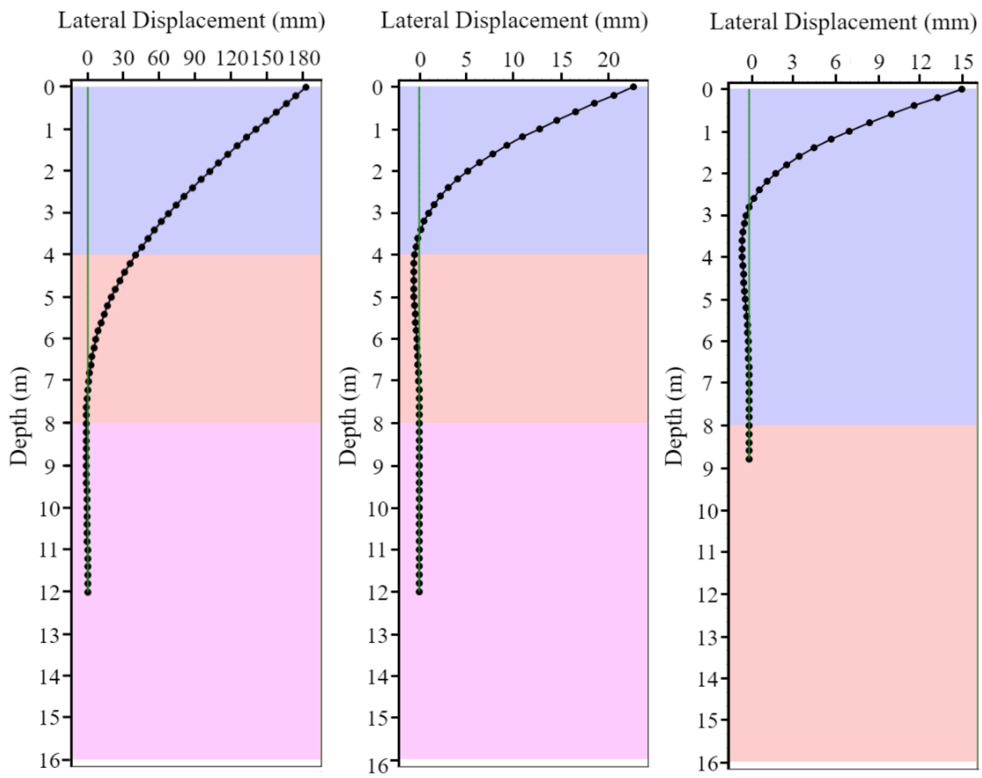


Figure 6. Maximum pile deflections for (a) Campinas-SP with clay p-y curves, (b) Campinas-SP with sand p-y curves and (c) Ilha Solteira-SP, determined by the nonlinear springs model (mm).

The maximum displacements of the soil obtained in PHASE² for Campinas-SP and Ilha Solteira-SP are displayed in Figures 7 and 8, respectively.

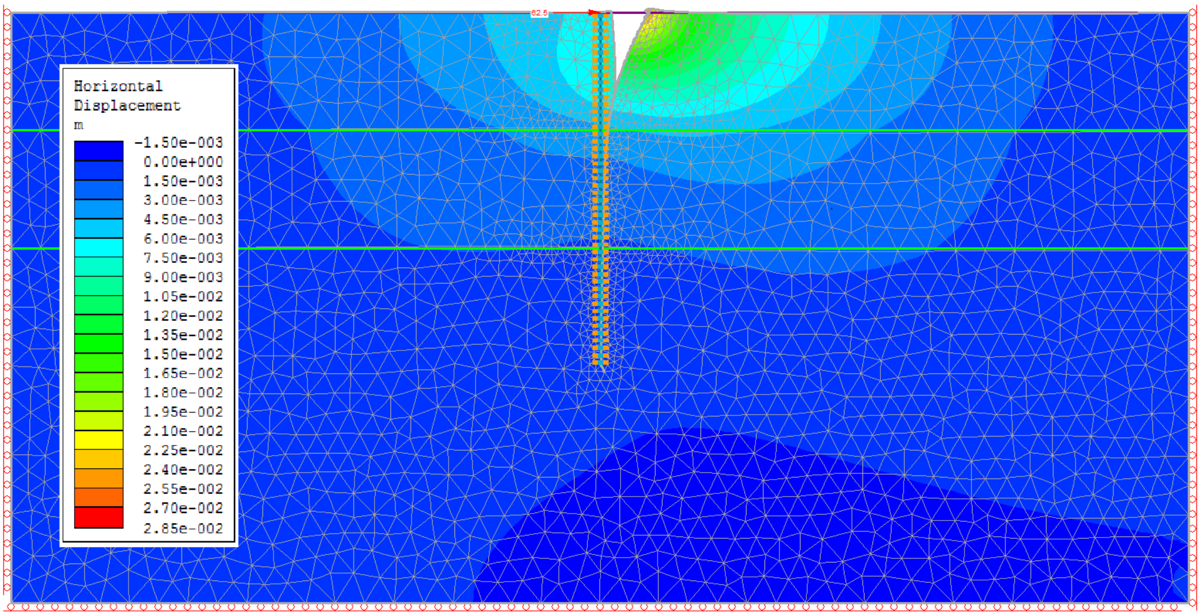


Figure 7. Maximum pile deflections for Campinas-SP, determined by the continuum model (m).

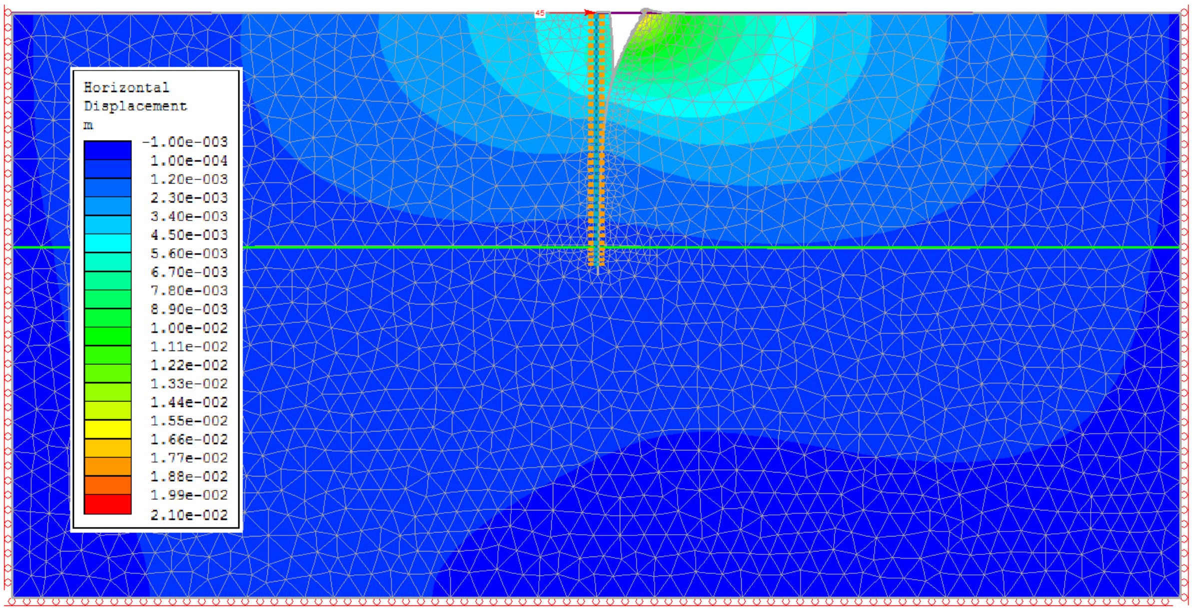


Figure 8. Maximum pile deflections for Ilha Solteira-SP, determined by the continuum model (m).

3.2 Graphics of Pile-top Displacements

The pile displacements were also taken progressively for each 5 kN of top-loading increment, so it was possible to generate evolutionary graphics of pile-top displacements against horizontal load applied, for each one of the employed methods. These curves, shown in Figures 9a and 9b, are then compared to the average displacements observed in the load tests realized by Miranda [15] and Del Pino [16], disregarding Pile 1 in each analysis.

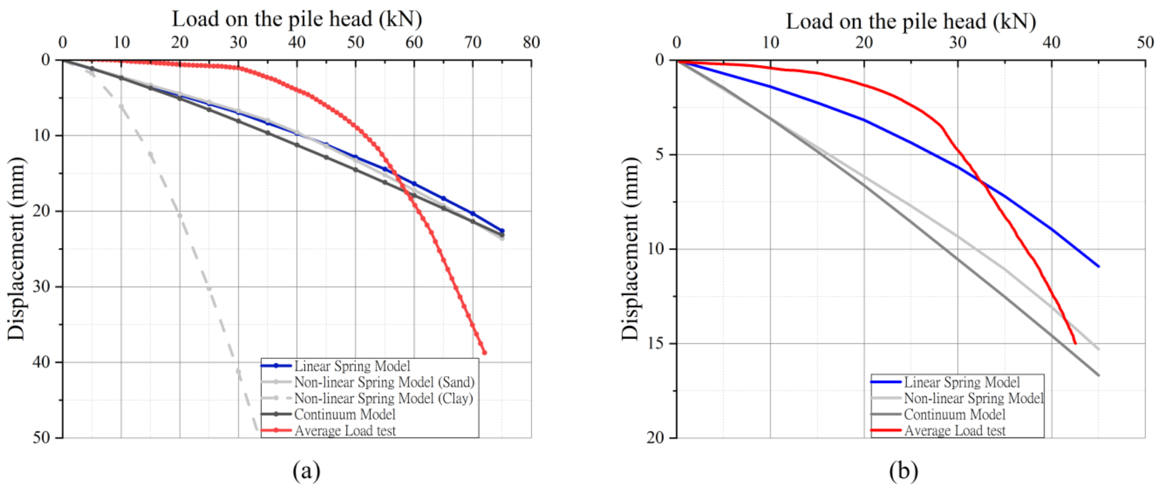


Figure 9. Evolutionary graphics of pile lateral load applied and top-pile displacements, for excavated piles in (a) Campinas-SP and (b) Ilha Solteira-SP.

3.3 Constants of Horizontal Subgrade Reaction

The equivalent values of η_h and T for each method in reference to soils in Campinas-SP and Ilha Solteira-SP are exhibited in Tables 7 and 8, respectively.

Table 7. Determination of the equivalent η_h and T, for excavated piles in Campinas-SP

Models	Corresponding load (kN)		η_h (MN/m³)	T (m)
	6 mm	12 mm		
Linear Springs	27,8	50,3	5,82	1,35
Nonlinear Springs (Clay)	10,3	14,6	0,90	1,97
Nonlinear Springs (Sand)	28,9	49,4	5,89	1,35
Continuum	22,3	40,2	4,02	1,46
Load Tests (Miranda, 2006)	45,6	53,0	11,55	1,18

Table 8. Determination of the equivalent η_h and T, for excavated piles in Ilha Solteira-SP

Models	Corresponding load (kN)		η_h (MN/m³)	T (m)
	6 mm	12 mm		
Linear Springs	33,1	50,4	11,22	1,01
Nonlinear Springs	21,2	39,9	6,50	1,13
Continuum	17,4	32,2	4,59	1,21
Load Tests (Del Pino, 2003)	32,9	38,4	8,32	1,08

It is noted that none of the estimates of T exceed 2 meters, which means the piles may be classified as long in all the conditions explored – as previously assumed –, since all tested piles have length over 8 meters.

The load test in Campinas showed an overall result of constant of horizontal subgrade reaction of around two times the equivalent ones obtained for most analysis methods. The methods provided very similar results, except for the nonlinear soft clay p-y curves model, which resulted in bigger displacements. In the soil of Ilha Solteira/SP, the methods provided more variety in η_h values, above and below the load tests results. The same deviation is observed for the continuum model, while the spring models resulted in better approximations.

4 Final Comments

The analysis developed in this research enables the evaluation of the most commonly applied methods of analysis for laterally loaded piles, in its linear and nonlinear phases, in comparison to load tests performed in typical soils of the State of São Paulo. Among the results presented, some important conclusions may be drawn:

- The values of η_h for load tests in Campinas-SP [17] resulted approximately two times bigger than the ones found with the chosen analysis methodologies, even though the three methods showed very similar results among them for both linear and nonlinear phases. There are several reasons for that, and one worth pointing out is the lack of investigation towards the Young's modulus of the piles, taken as a default value of 21000 MPa. This is an inaccurate procedure, once the deformability of the set is just as depending of the flexural stiffness of the pile [EI], as it is of the stiffness of the upper soil layers. There also might be a natural divergence between the studied terrain and the soils to which the methods were developed.

- For the soil in Ilha Solteira-SP [18], a similar reason of 2 was observed for values of η_h obtained with the continuum models, whereas the linear and nonlinear spring models resulted in better approximations. For these conditions, the values of η_h should have been taken with smaller displacements, since the load tests indicate the pile-soil set has already advanced to the plastic phase with a 12 mm displacement. When analyzing the pile-top displacements, the analysis errors in all cases are on the safety side for the elastic part and become unsafe in the final plastic phase, developing smaller displacements than the experimental tests.

- For both cases studied, the lateral load tests indicated a slow initial deformation process in the elastic phase, and then a rapid yielding process at the final loading phase. This behavior isn't precisely simulated by the discrete models, which develop a much more gradual process of yielding. The main reason is the fact that these researched methods take in consideration the concrete as an elastic material, while the foundation piles tested have definitely suffered from fissures and cracks over higher loads, losing part of their flexural stiffness. This reveals the importance of accounting concrete as an elastoplastic material, especially when the design requires the verification of greater displacements. Models which apply loads in iterative steps, such as the nonlinear spring model and the continuum model, also facilitate the considering of concrete's nonlinearity.

- The linear spring model with a plasticity criterion provided decent results and demonstrated an advantage on the other methods over it simplicity of application. For this work, a program was developed to automatically estimate the rupture of linear springs and replace them with forces, but this procedure can also be manually done with standard FEM software. The other two methods, on the other hand, require the use of specific geotechnical software or complex programming. If the initial stiffness of the soil and the concrete's parameters are taken with precision, the linear spring model has very good potential, particularly when the applied loadings and displacements are not too high. This method is also easily applied for bigger and more complex models, when the foundation is not isolated from the superstructure.

- When the nonlinear spring model is employed, there might be some confusion over which p-y curve model to choose, especially if the soil is too variegated. The analysis for Campinas-SP demonstrates that an inaccurate choice of the soil model can promote expressive changes in results, since the model with p-y curves for clays predicted displacements of almost 8 times the ones with sand p-y curves. With the linear spring or continuum models, this problem is avoided, seeing that these methods include a bigger variety of soils types and that this kind of input data has lower impact on the results when compared to the resistance and deformability parameters of the soil (φ, c, E, ν and N_{SPT}).

- The use of the continuum model requires some special care regarding the interaction between soil and structural elements, particularly if a 2D plane-strain model is employed. A rupture pattern at the interface was provided to allow the opening of a gap between the soil and the tensioned soil, based on the Mohr-Coulomb model. Besides, the design must consider the pile actions form larger bulbs of pressure in the soil, which was done by considering a spreading factor of 3 and adjusting loads and dimensions to a 1 meter column of soil.

- The choice of an analysis method for laterally loaded piles must always consult the amount and level of quality of the geotechnical information available. For instance, there is a long historical practice of Standard Penetration Test related parameters in Brazil, while the use of parameters such as the strain corresponding to one-half the maximum principal stress difference (ε_{50}), used in the nonlinear analysis, is not commonly obtained for soils in the same area. In these cases, the engineer is usually forced to resort on not so reliable empirical correlations, what might add uncertainties and compromise the safety of the structural design.

References

[1] D. A. Velloso and F. R. Lopes. *Fundações: Critérios de Projeto, investigação do Subsolo, Fundações Superficiais e Fundações Profundas*. Oficina de Textos, 2011.
[2] E. E. De Beer. Piles Subjected to Static Lateral Loads. *IX ICSMFE*. Tokio, 1977.
[3] H. Matlock, L. C. Reese. Generalized Solutions for Laterally Loaded Piles. *Journal of SMF Division*. October-1960.
[4] D. A. Velloso, P. E. L. Santa Maria and F. R. Lopes. *Princípios e Modelos Básicos de Análise*. In: W. Hachich et al. Fundação: Teoria e Prática. Pini Ltda., 2016.
[5] E. Winkler. *Die Lehre von der Elastizitat und Festigkeit.* Dominicus, 1867.
[6] American Petroleum Institute (API). *Recommended practice for planning, designing, and constructing fixed offshore platforms*, Section 6.8 Soil Reaction for Laterally Loaded Piles. Washington D.C., United States, 2010.
[7] M. Huang, X. Zhou, J. Yu, C. F. Leung and J. Q. W. Tan. Estimating the effects of tunnelling on existing jointed pipelines based on Winkler model. *Tunnelling and Underground Space Technology*, vol 86, pp. 89–99, 2019.
[8] G. Anoyatis and A. Lemnitzer. Kinematic Winkler modulus for laterally-loaded piles. *Soils and Foundations*, vol 57, pp. 453–471, 2017.
[9] C. Zhang, P. Deng and W. Ke. Assessing physical mechanisms related to kinematic soil-pile interaction. *Soil Dynamics and Earthquake Engineering*, vol 114, pp. 22–26, 2018.
[10] A. M. Hassan. Winkler model for pile seismic analysis considering end constraints effects. *HBRC Journal*, vol 14, pp. 316–320, 2018.
[11] A. Rahmani et al. Evaluation of p-y springs for nonlinear static and seismic soil-pile interaction analysis under lateral loading. *Soil Dynamics and Earthquake Engineering*, vol. 115, pp. 438-447, 2018.
[12] W. Zhanga, E. E. Seylabib and E. Taciroglua. An ABAQUS toolbox for soil-structure interaction analysis. *Computers and Geotechnics*, vol. 114, 2019.
[13] P. Ni et al. On predicting displacement-dependent earth pressure for laterally loaded piles. *Soils and Foundations*, vol. 58, n. 1, pp.85-96, 2018.
[14] Associação Brasileira de Normas Técnicas (ABNT). *NBR 6122: Projeto e execução de fundações*. Rio de Janeiro, Brazil, 2010.
[15] G. Miranda Jr. *Estacas Submetidas a Esforços Horizontais em Solos Colapsíveis do Interior de São Paulo, nas Condições Natural, Melhorada e Inundada*. PhD thesis. State University of Campinas, 2006.
[16] A. Del Pino Jr. *Análise do Comportamento de Estacas do Tipo Broca Escavada com Trado Mecânico, Solicitadas por Esforços Transversais*. PhD thesis. São Paulo State University, 2003.

[17] P. J. R. Alburquerque. *Estacas Escavadas, Hélice-Contínua e Ômega: Estudo do Comportamento à Compressão em Solo Residual de Diabásio, através de Provas de Carga Instrumentadas em Profundidade.* PhD thesis. University of São Paulo, 2001.
[18] A. H. Teixeira, N. S. Godoy. *Análise, Projeto e Execução de Fundações Rasas*. In: W. Hachich et al. Fundação: Teoria e Prática. Pini Ltda., 2016.
[19] J. E. Bowles. *Foundation Analysis and Design*. McGraw Hill Book Company, 1982.
[20] K. Terzaghi. Evaluation of Coefficients of Subgrade Reaction. Geotechnique, *The Institution of Civil Engineers*. London, v.5, n.4. 1955 p.297-326.
[21] N. Aoki and D. Velloso, 1975. An approximate method to estimate the bearing capacity of piles. In: V *Congresso Panamericano de Mecânica dos Solos e Engenharia de Fundações*, pp. 367–376.
[22] R. Marche. *Estimated Resistance and Soil-Pile Interaction in view of Bending Moments*. École Polytechnique de Montreal, Canada, 1974.
[23] W. J. M. Rankine. On the stability of loose earth. *Philosophical Transactions of the Royal Society of London*, vol 146, pp. 9–27, 1857.
[24] A. L. Bell. The lateral pressure and resistance of clay and the supporting power of clay foundations. *Minutes of the Proceedings of Institution of Civil Engineers*, vol. 199, pp. 233–272, 1915.
[25] Rocscience Inc. *RSPile Model Program Reference Manual*. Toronto, Canada, 2018.
[26] L. C. Reese, W. Van Impe. *Single Piles and Pile Groups Under Lateral Loading*, 2nd ed., pp 53-118, 2011.
[27] R. B. Peck, W. E. Hanson, T. H. Thorburn. *Foundation Engineering*, 2nd ed, New York, 1974.
[28] RocScience Inc. *Phase² Model Program Reference Manual*. Toronto, Canada, 2001.
[29] PLAXIS. *Plaxis 2D Anniversary Edition: Scientific Manual*. Delft, Netherlands, 2014.