Efficiency Evaluation of Micropiles Settlement Estimation Methodologies on Silty-Sand Soil

Fernando Feitosa Monteiro (1); Marcos Fábio Porto de Aguiar (2); Renato Pinto da Cunha (3); Leila Maria Coelho de Carvalho (4)

(1) Ph.D Candidate, University of Brasília, Brazil, engffmonteiro@gmail.com
(2) Ph.D in Geotechnical Engineering, Federal Institute of Ceará, Brazil, *marcosfpa@hotmail.com*
(3) Ph.D in Geotechnical Engineering, University of Brasília, Brazil, rpcunha@unb.br
(4) MS.c Candidate, University of Brasilia, Brazil, *leila.mariacarvalho@hotmail.com*

Abstract

The design of deep foundations in Brazil is often based on field investigations, where SPT (Standard Penetration Test) tests are the only ones to be performed, often recurring to correlations in order to obtain resistance parameters, thus enabling, settlement estimations of deep foundation foundations from theoretical, empirical and numerical methods. Afterwards, load tests were carried out in order to verify the predicted behavior. The main objective of this paper is to analyze the efficiency of semi-empirical methods for micropiles settlement prediction from standard penetration tests results through a comparison with load tests results and numerical simulations. In this work, the settlement of a micropile with 0,31 m diameter and 6 m length was evaluated through empirical methodologies using standard penetration tests parameters. Thus, evaluating the efficiency of settlement estimations using empirical methods, for regions that have silt-sandy soils, improving predictions adopted in deep foundations design. The research verified that the empirical methods presented inconsistent estimates in the prediction of deep foundations settlement on silt-sandy soil of the city of Fortaleza, since settlement estimates using empirical methods presented higher values than those obtained in load tests. In the study it is verified the importance of a satisfactory load capacity estimate, due to its influence in the prediction of the evaluated settlements.
Keywords: Micropiles; Numerical Modeling; Settlement;

1 Introduction

Adequate knowledge of the soil is indispensable for foundation execution. Geotechnical investigations are essential for obtaining soil parameters. Thus, the various layers that compose the subsoil are identified, classified and evaluated. The foundation project design is of predominant importance, as the entire load of the structure is transferred to the foundation and subsequently to the soil. In Brazil, foundation projects are often designed from Standard Penetration Tests (SPT) and complementary tests are usually not performed, making the use of correlations to obtain soil resistance parameters. The geotechnical design of deep foundations considers, for the most part, only the pile load capacity. However, it is worth emphasizing the importance of the settlement analysis, being this a primordial aspect to assure structural security of the foundation element and the whole superstructure. There are several theoretical, semi-empirical, empirical and numerical methods for the estimation of the settlements in deep foundations used in Brazil and in the world. Thus, the present work has the aim to evaluate settlement estimation of deep foundations, comparing the obtained results with load tests and numerical simulations.

2 Pile load capacity and settlement

According to [1], semi-empirical methods can be defined as those based from established theoretical formulations, but that allow to estimate the skin friction resistance and tip resistance from correlations with field tests parameters. According to the Brazilian standard [2], semi-empirical methods are those in which the properties of materials, estimated on the basis of correlations, are used in adapted theories of soil mechanics.

According to [3], the load capacity of micropiles with a diameter (B ≤ 0,45m) and injection pressure (p ≤ 400 kPa) can be estimated with:

$$Q_{ult}=(\beta_0\beta_2N_b)A_b + \mathrm{U}\sum(\beta_0\ \beta_1\,\mathrm{N})\ \Delta\mathrm{L} \quad (1)$$

Where: ΔL is the soil thickness characterized by a given N_{SPT}, Nb is the N_{SPT} at the tip level, U is the pile perimeter, β1 and β2 are factors depending on the soil type and βo is a factor that depends on the pile diameter (B) and injection pressure, which can be calculated by the following equation:

$$\beta_0 = 1 + 0{,}11p - 0{,}01\,B \quad (2)$$

[4] does not take into account the tip resistance, considering only the skin friction resistance. The author proposes that the ultimate load capacity of the micropile is given by:

$$Q_{ult} = \pi dLKI \quad (3)$$

Where: d is the pile diameter; K is the coefficient that represents the average pile-soil interaction that is, the soil-pile adhesion (Table 1); L is the pile length; I is the dimensionless shape coefficient, which depends on the diameter of the pile (Table 2).

Table 1 – Values of K

Soil characteristics	kPa
Soft	50
Loose	100
Medium compact	150
Very compact (gravel and sand)	200

Table 2 – Values of I

Pile diameter (mm)	I
100	1,00
150	0,90
200	0,85
250	0,80
310	0,74
410	0,64

The performance of a civil engineering work is related to the degree of alteration of the soil mass during the execution phase. According to [2], a foundation project must meet the following basic requirements: adequate safety to soil collapse, structural elements, and acceptable deformations under working conditions. These requirements are met by checking the service limit state (SLS) and ultimate limit state (ULS). According to [5], pile settlement can be calculated in three stages, where in the first stage the average axial force in each segment of length ΔL, pile section area (A_{med}), and pile elasticity modulus (E_p) are computed, thus:

$$\Delta H_{s,s} = \frac{P_{med}}{A_{med}} \frac{\Delta L}{E_p} \tag{4}$$

In the second stage, the settlement on the pile tip is computed as:

$$\Delta H_{pt} = \Delta q \; D \, \frac{1-\mu^2}{E_s} \, m \, I_s I_f F_1 \tag{5}$$

Where: m I_s = 1 (form factor); If is the embedding factor (If = 0.55 if L / D ≤ 5; If = 0.5 if L / D> 5); D is the pile diameter or the smaller pile dimension; μ is the poisson coefficient (the author suggests to use a value of 0.35); Δq is the load on the foundation; E_s is the soil elasticity modulus below the pile tip, and can be obtained by the following relationships: SPT - E_s = 500 (N + 15) in kPa and CPT - Es = 3 to 6 q_c (use values of 5.6 if OCR is greater than 1) in kPa; F1 is the reduction factor ranging from 0.25 if lateral resistance reduces the tip load Pp ≤ 0; 0.5 if the load at the tip Pp> 0; 0.75 if there is only tip load. The factor F1 is used by the ratio of the tip region to move down due to the load on the tip and the tip rebound due to lateral resistance along the shank pulling the soil-foundation system down. This method uses the total axial load, which is known, and the factor F1, which is an estimated value. In the third stage, the soil and the pile settlement are added, resulting in the total pile settlement.

[6] proposes a methodology for the prediction of the load-settlement curve of a pile foundation, known a point on this curve and considering Van der Veen's (1953) expression [7]:

$$P = R\,(1 - e^{-\alpha\,\rho}) \tag{6}$$

Where the parameter α defines the shape of the curve, which can be determined by:

$$\alpha = \frac{-\ln(1 - \frac{P}{R})}{p} \tag{7}$$

Thus, by calculating the load capacity R and estimating the settlement (ρ), for a load (P), one can determine the value of α.

3 Case Study

The case study was carried out in the foundation experimental field of the University of Fortaleza (UNIFOR), located in the city of Fortaleza, in northeastern Brazil. In the study area, 4 reaction piles and 1 micropile were performed using 1: 1,2: 0,5 (cement, sand, water) mortar ratio and with an injection pressure of 300 kPa. The mortar was casted with the aid of a mixer. The analyzed micropile presents a length of 6 m and a diameter of 0,31 m. Table 3 presents the results of soil characterization tests performed in the UNIFOR experimental field. It is verified that the statigraphy of the experimental field is composed in its totality of a non-plastic silty sand soil.

Table 3 – Soil Characterization

Depth (m)	Soil Texture (%)					LL	PL	Specific Gravity
	Gravel	Coarse sand	Medium sand	Fine sand	Silt and Clay			
1	0	0,57	15,49	67,83	16,11	-	NP	2,62
2	0	0,27	16,71	61,87	21,15	-	NP	2,63
3	0	0,34	13,75	67,00	18,91	-	NP	2,59
4	0	0,46	15,58	63,34	20,62	-	NP	2,60

In the case study, the SPT test was performed according to [8], using the bentonite sludge during the drill execution so that it could reach the impenetrable stratum. Figure 1 presents the SPT test results, as well as the length of the pile, the diameter and the general stratigraphy of the soil. It is verified that the soil stratigraphy is composed of two types of soils, a 6m thick silty sand layer and a 14 m clayey silt layer. The penetration resistance index values (N_{SPT}) presented low values up to 9 m depth. The presence of the water table on a 5m depth was identified.

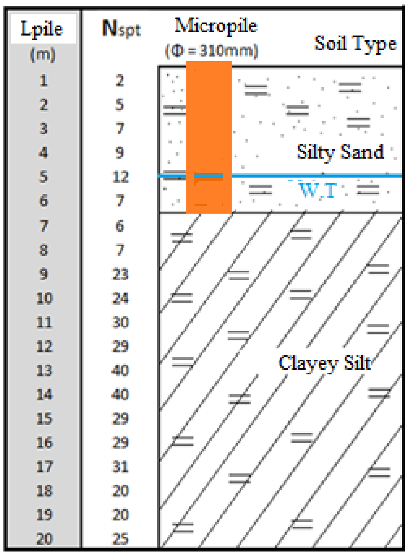

Fig. 1 – SPT Results

The pile was subjected to a static load test, according to [9]. The test was carried out in 10 load stages, each stage corresponded to 20% of the pile work load. Figure 2 presents the load-settlement curve obtained during the pile load test. The pile was subjected to a maximum load of 500,85 kN, reaching a maximum settlement of 35,57 mm. After unloading, a residual settlement of 32,74 mm is observed.

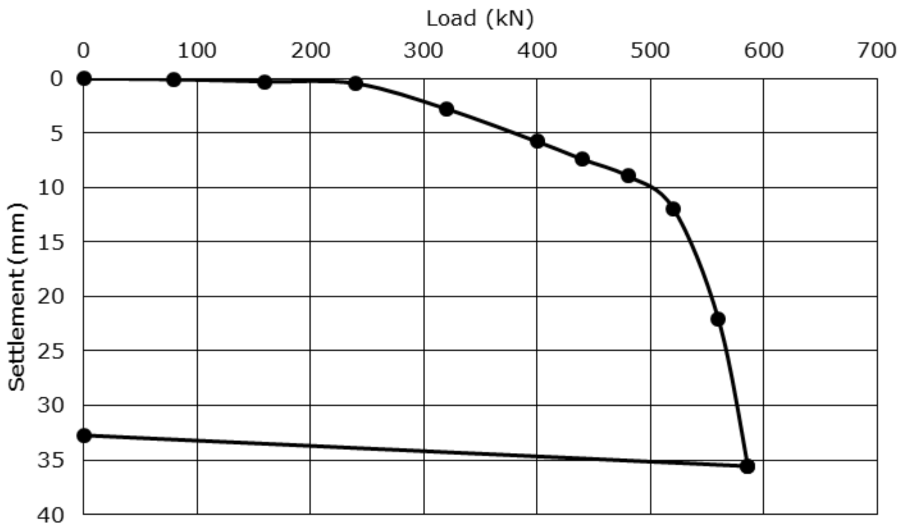

Fig. 2 – Load-Settlement Curve

The load test was simulated in the Plaxis 2D software, the numerical analysis was performed in 2 stages, the first stage simulated the pile construction and in the second stage, the loading was evaluated. Due to the case studied, an axissimetric analysis was used to gain computational time in the simulations, 15-node elements were chosen. For the pile simulation, a length of 6 m and a diameter of 0,31 m was adopted. A modulus of elasticity of 22 GPa was adopted, according to the values indicated by [10] for the water-mortar ratio. For the implementation of soil resistance parameters, it was necessary to use semi-empirical correlations using N_{SPT}, thus estimating values for cohesion, friction angle and soil deformability modulus as a function of depth. The equations used to obtain the parameters are shown below:

$$\varphi = \sqrt{20 + N_{SPT}} + 15^0 \quad (8)$$

$$C = 10\ N_{SPT} \quad (9)$$

$$E_s = 3500\ N_{SPT}\ (kPa) \quad (10)$$

In terms of pile load capacity, the undrained condition usually predominates as critical, as the load capacity tends to increase with the dissipation of the porepressures. Therefore, it is usual to calculate the load capacity only with undrained values of cohesion and friction angle. Thus, allowing the estimation of the soil parameters to perform the numerical simulation, as shown in Table 4. For each soil layer, the average N_{SPT} was used to apply the semi-empirical correlations, in order to determine the shear strength parameters. Due to the lack of laboratory tests, the use of N_{SPT}-related correlations is necessary to obtain shear strength parameters values to be used in numerical analyzes. Occasionally, this methodology presents shear strength parameters values that could be considered incoherent for the simulated soil type. Usually, an adjustment is made, and a coherent value is adopted according to the soil type. But in this paper, the obtained values from the N_{SPT} correlations (equations 8, 9 and 10) were used without an adjustment, in order to evaluate the efficiency of those correlations. Figure 3 shows the numerical simulation geometry in the Plaxis 2D software.

Table 4 – Shear Strength Parameters

Material	**Model**	**Unit Weight (kN/m³)**	**Friction Angle (°)**	**Cohesion (kPa)**	**Elasticity Modulus (kPa)**	**υ**
Silty Sand	Mohr-Coulomb	18	27	70	24500	0,3
Clayey Silt	Mohr-Coulomb	21	37	250	75643	0,4
Pile	Linear-Elastic	24	x	x	22000000	0,15

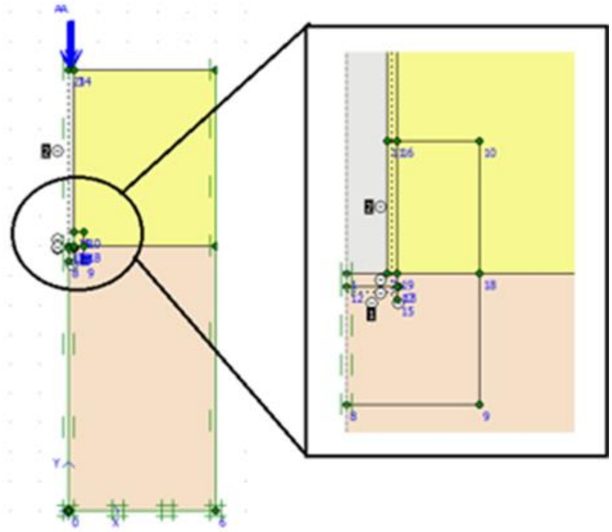

Fig. 3 – Numerical Simulation Geometry

4 Results and Discussions

Table 5 shows the estimated load capacity values for the methods of [3] and [4]. Then, the load capacity data are used for settlement computation using the [5] method, as seen in Table 6.

Table 5 – Load Capacity from Semi-Empirical methods

Method	Load Capacity (kN)
Cabral (1986)	484,6
Lizzi (1982)	701,2

Table 6 – Settlement from Semi-Empirical methods

Method	Cabral (1986)	Lizzi (1982)
Pile settlement (mm)	0,84	0,18
Soil settlement (mm)	41,59	60,17
Total settlement (mm)	41,77	60,35

According to Table 6, it is observed that the values of pile settlement are much lower than the values of pile settlement, having this greater relevance in the value of the total settlement. Finally, equation 6 was used to predict the load-settlement curve, so a load greater than the skin friction resistance and less than or equal to the failure load was adopted. A value of α (factor that defines the shape of the curve) is found, thus, the values of the settlement are estimated and the loads corresponding to these settlements are obtained, as shown in Table 7, resulting in the mathematical expression of the load-settlement curve.

Table 7 – Comparative settlement results

Load Test		PLAXIS 2D		Lizzi (1982)		Cabral (1986)	
Load (kN)	Settlement (mm)	Load (kN)	Settlement (mm)	Load (kN)	Settlement (mm)	Load (kN)	Settlement (mm)
0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
80,0	0,1	71,6	1,4	190,3	3,0	181,7	3,0
160,0	0,3	179,2	3,5	329,0	6,0	295,3	6,0
240,0	0,4	215,0	4,3	430,0	9,0	366,3	9,0
320,0	2,8	250,8	5,0	503,6	12,0	410,7	12,0
400,0	5,8	286,7	5,8	557,3	15,0	438,4	15,0
440,0	7,4	321,0	6,6	596,3	18,0	455,8	18,0
480,0	8,9	458,0	15,3	624,8	21,0	466,6	21,0
520,0	11,9	550,6	29,9	645,5	24,0	473,4	24,0
560,0	22,1	585,8	36,6	660,6	27,0	477,6	27,0
585,9	35,6	600,5	39,8	671,7	30,0	480,3	30,0
-	-	629,6	46,2	683,8	35,0	482,7	35,0

It was verified that the curves presented similar behaviors up to a load of 100 kN. For the pile work load, which is 300 kN, a 2,5 mm settlement is verified, while for the semi-empirical methods, settlement estimates of 5,1 and 7,3 mm for Lizzi (1982) and Cabral (1986) methods, respectively, although the methods present larger values when compared with the reference values, it is verified the same order of magnitude. As for the load capacity, a difference of 17.2% is observed between the load test and the Cabral (1986) method, which is a conservative method in favor of safety. On the other hand, Lizzi's method (1982) was shown to be unsafe because it presented a higher load capacity than the load test result, being that difference around 16.5%.

Table 7 presents the values of the load-settlement curves for the different methodologies used in this paper and Figure 4 presents the load-settlement curve obtained by the same methodologies. It can be seen that the numerical modeling showed agreement with the values of the semi-empirical methods up to a load of 400 kN. The numerical simulation estimated a settlement of 6,2 mm for a work load of 300 kN, while the load test presented a settlement of 2,5 mm, thus verifying that the numerical simulation presented a settlement value 3 times higher than observed in the load test. It is worth mentioning that, although the numerical simulation presented a higher settlement than load test, the order of magnitude of the values obtained is similar, and from the point of view of the foundation engineering, it can be inferred that the settlement calculated by the numerical simulation is in agreement with the settlement value obtained by the load test. Comparing the settlements obtained by semi-empirical methods, convergent settlement values are observed. Evaluating the load-settlement curves behavior, it is possible to verify that the numerical simulation was the methodology that presented the best load-settlement behavior estimate. While the semi-empirical methods presented significant divergences when simulating the behavior of the load-settlement curve when compared with the load test.

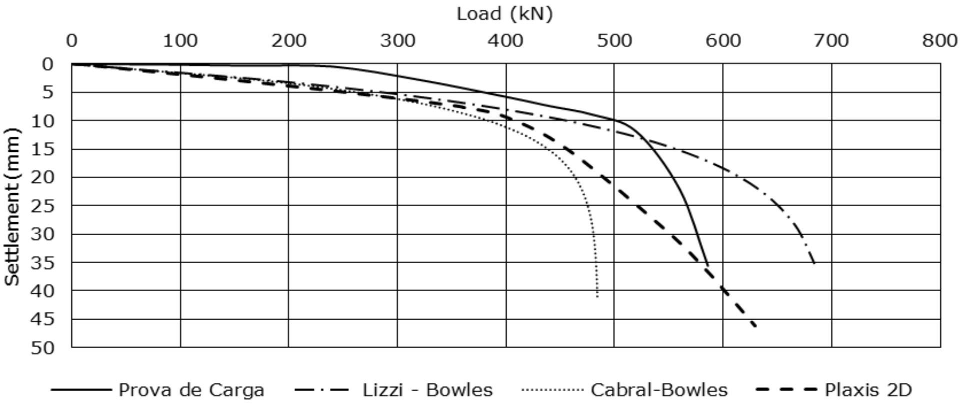

Fig. 4 – Methodologies comparison

For the failure load of 585,9 kN, the numerical simulation presented a 36,6 mm settlement, while the load test presented a settlement value of 35,6 mm. Thus, indicating that the numerical modeling presented converging values when compared with the load tests for the failure load. It is worth mentioning that the parameters used for the numerical simulation were obtained from N_{SPT} correlations, a fact that may explain the agreement of the simulation with the settlement semi-empirical estimation methods for the analyzed work load.

5 Conclusions

The SPT test is a simple and low cost geotechnical investigation method, that has some limitations that can generate a great diversity in the results of calculations based on its data. Because of this, it has been found that the settlement estimation micropiles through methods based on SPT tests can provide different settlements estimates than those obtained in load tests, which are measurements considered as reference in foundations projects.

It was possible to verify that the semi-empirical methods presented reasonable estimates from the engineering point of view. However, from the numerical point of view, they presented higher values in magnitude order 2 or 3. Although the semi-empirical methods presented higher settlement estimates, it was observed the same order of magnitude. It is also worth mentioning the importance of a reasonable load capacity estimate, since it influences directly the settlement estimation. A lower value of 17.2% was found when comparing the Cabral method (1986) and the load test regarding the load capacity estimation; whereas the Lizzi (1982) method presented a higher load capacity estimation when compared with load test, overestimating the micropile load capacity.

Numerical modeling using NSPT-based parameters showed agreement with the values of the semi-empirical methods up to a load of 400 kN. The numerical simulation estimated a 2,5 higher settlement than that obtained by the load test when comparing the settlements for the same work load. In spite of this, it is possible to infer that the result of the numerical analysis presented a satisfactory estimate, considering that the settlement values were of the same order of magnitude.

Acknowledgments

The authors would like to thank FAP-DF (Federal District Foundation for Research Support) for funding the participation in the event.

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