

Forecasting bearing capacity performance with semi-empirical and theoretical methods applied to precast concrete piles founded on sandy clay in the region of Uberlândia-MG, Brazil

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Abstract

This article presents a study on the performance of semi-empirical methods based on the Standard Penetration Test (SPT) for the prediction of bearing capacity already disseminated in the practice of Brazilian Foundation Engineering (Aoki and Velloso, 1975; Décourt and Quaresma, 1978, 1996; Teixeira, 1996), together with the recent method proposed by Pereira (2020), and the theoretical method known as α Method (disseminated around the world). These were applied to precast concrete piles based on sandy clay in the Uberlândia-MG region. In the performance analysis, the ultimate shaft and tip resistances and the bearing capacity values, which were utilized as references, were mobilized in the dynamic loading tests performed on ten piles. The results of these tests were compared with those obtained using the mentioned methods. In general, both methods yielded robust results, with relatively accurate predictions, except the α Method, which was found to perform below the semi-empirical methods considered. The research still highlights the need for discretion in the application of semi-empirical/theoretical methodologies because there are certain scenarios wherein these methods can yield inaccurate results.

Keywords: pile foundation, bearing capacity, semi-empirical/theoretical methods, SPT, dynamic load testing.

1. Introduction

The technical field of foundation engineering comprises a series of methodologies for calculating the geotechnical load capacity of deep foundations. However, calculating this load is challenging, since it requires an evaluation of the soil-pile load transfer mechanism (soil-structure interaction) prior to the calculation. Considering the significance of obtaining the geotechnical load capacity, it was found that it is a function of a series of variables such as the executive methodology of the pile, type of soil along the shaft and at the tip, and geometric parameters. Therefore, semi-empirical methods of estimating geotechnical load capacity based on data from the standard penetration test (SPT) and theoretical methods are considered.

Velloso and Lopes (2010) reported that in Brazil, the drilling SPT is the most frequently performed geotechnical investigation. Milititsky (1986) stated that “Foundation engineering in Brazil can be described as the Geotechnics of SPT.” Therefore, foundation engineers have been focusing on establishing methods for calculating the load capacity of piles using the results of percussion drilling.

2. Background

2.1 Geotechnical load capacity of a foundation pile

The geotechnical load capacity represents the threshold of the system’s failure formed by the structural element (pile) and the geotechnical mass surrounding the pile (Cintra and Aoki, 2010); it is the value of the force corresponding to the maximum

resistance that the system can offer at failure, from a geotechnical perspective. The general expression (Equation 1) of the load capacity (R) of the pile foundation elements includes two sets of variables: (1) the pile geometric variables denoted by the

According to Schnaid and Odebrecht (2012), semi-empirical methods are valuable tools in the field of foundation engineering. However, they highlight that it is important to recognize that their validity is limited to the specific conditions of the historical cases used in their development. Moreover, Alledi (2013) reported that these methods are mainly used to estimate the length of the piles and consequently, estimate project costs. Theoretical methods are methods that use the theoretical mathematical model for the bearing capacity of an isolated pile element associated with soil resistance parameters (cohesion and friction angle). Cintra and Aoki (2010) highlight that such methods constitute a vast chapter of Foundation Engineering. Pérez (2014) reinforces that, due to the limitation of the theoretical model that considers the soil as cohesive or granular, and, in addition, the fact that the theoretical formulas of deep foundation load typically does not provide reliable predictions, authors such as Cintra and Aoki (2010) recommend, in the practice of foundation projects by piles, the restricted use of theoretical formulas.

Foundation engineers are responsible for determining the geotechnical parameters of the soil relevant to the semi-empirical and theoretical methods, to predict the geotechnical load capacity of foundation elements. This theoretical geotechnical load capacity should preferably be validated in the field, especially via static testing and/or dynamic load testing (DLT), according to the guidance of NBR 6122 (ABNT, 2019).

Therefore, this study aims to evaluate the performance of semi-empirical methods based on SPT for estimating the bearing capacity widely disseminated in the practice of Brazilian Foundation Engineering - Aoki and Velloso (1975), Décourt and Quaresma (1978, 1996) and Teixeira (1996) - and the recent method proposed by Pereira (2020) and the theoretical method known as α Method (disseminated around the world) both applied to precast concrete piles founded on sandy clay in the Uberlândia-MG region. For the performance analysis, the values obtained via DLT were considered as the reference, and these results were compared to the values obtained using the mentioned methods.

2.2 Semi-empirical methods

Amann (2010) defines semi-empirical methods as methods that start from theoretical formulations and are complemented with the establishment of the maximum friction and tip stresses obtained via em-

pirical correlations with the field tests. Therefore, surveys are an essential part of semi-empirical methods.

Ever since the semi-empirical methods proposed by Aoki and Velloso (1975), the

perimeter (U), length of the pile segment (ΔL), and cross-sectional area of the pile tip (A_p) and (2) the geotechnical variables denoted by the stress mobilized at the tip (r_p) and the shaft stresses mobilized in the segments constituting the pile shaft (r_L).

$$R = U \cdot \sum (r_L \cdot \Delta L) + r_p \cdot A_p \quad (1)$$

2.2.1 Pereira’s method (2020)

The method proposed by Pereira (2020) is valid for precast concrete piles

on sandy clay. This method was developed based on the results of DLT and SPT type

surveys. Based on this method, the load capacity is expressed as follows:

$$R = (3.73) \cdot U \cdot L \cdot \bar{N}_L + (192.16) \cdot A_p \cdot \bar{N}_p \quad (2)$$

where L is the pile length, and N_L is the average number of strokes achieved over the entire length of the pile; when calculating \bar{N}_L , the values used in the

evaluation of tip resistance are not considered. \bar{N}_p is the average value of the index of resistance to the penetration in the pile tip, obtained from three

values—the one corresponding to the level of the tip, one immediately before 1 m above, and the one immediately after 1 m below.

2.3 Theoretical method - α Method

The α Method is based on the undrained shear strength of cohesive soils. In this method, the shaft resistance

and the tip resistance are assumed to be proportional to the undrained shear strength (s_u). According to Murthy

(2007), the tip resistance is determined as Equation 3:

$$R_p = 9 \cdot s_u \cdot A_p \quad (3)$$

Via the α Method, the shaft resistance is calculated by Equation 4:

$$R_L = \alpha \cdot s_u \cdot U \cdot \Delta L \quad (4)$$

where α is the adhesion coefficient on the interface between soil and pile.

Kulhawy and Phoon (1993) propose that the following correlation for α :

$$\alpha = 0.5 \left(\frac{p_a}{s_u} \right)^{0.5} \quad (5)$$

where p_a is the atmospheric pressure, assumed by authors to be approximately 100 kPa.

2.4 Correlations with SPT data for obtaining the undrained shear strength (s_u)

In the absence of specific tests to determine the undrained resistance to

shear (s_u), the following correlations are suggested based on SPT data:

Teixeira and Godoy (1996):

$$s_u = 10 \cdot N \quad (6)$$

Hara *et al.* (1974):

$$s_u = (29.7) \cdot N_{60}^{0.72} \quad (7)$$

Kulhawy and Mayne (1990):

$$s_u = (0.06) \cdot p_a \cdot N_{60} \quad (8)$$

Terzaghi and Peck (1967):

$$s_u = 6 \cdot N_{60} \quad (9)$$

where: p_a is the atmospheric pressure, assumed to be approximately 101.3 kPa; N corresponds to the value

of the index of resistance to the penetration considered without the energy specification of the SPT; N_{60} corresponds

to the value of the index of resistance to the penetration, considering the energy to be 60% of the SPT.

3. Case study

The example considered in this study is a multipurpose building with an access

ramp (Figure 1), located in the city of Uberlândia-MG.



Figure 1 - Side view of the building (Mesquita, 2019).

The building is founded on 201 precast prestressed hexagonal concrete piles of full section (filled) termed as P27, P31, and P34. In such a denomination, the letter P indicates the polygonal pile, and the number in the sequence

indicates the diameter of the piles in centimeters. For the design of the access ramp foundations, 48 precast circular reinforced concrete piles with hollow sections, an external diameter of 42 cm, and an internal diameter of

25 cm were adopted. Figures 2 and 3 present the floor plan of the foundations, indicating the points where the SPT surveys were performed (SP-XX) and the crown blocks whose piles were subjected to the DLT.

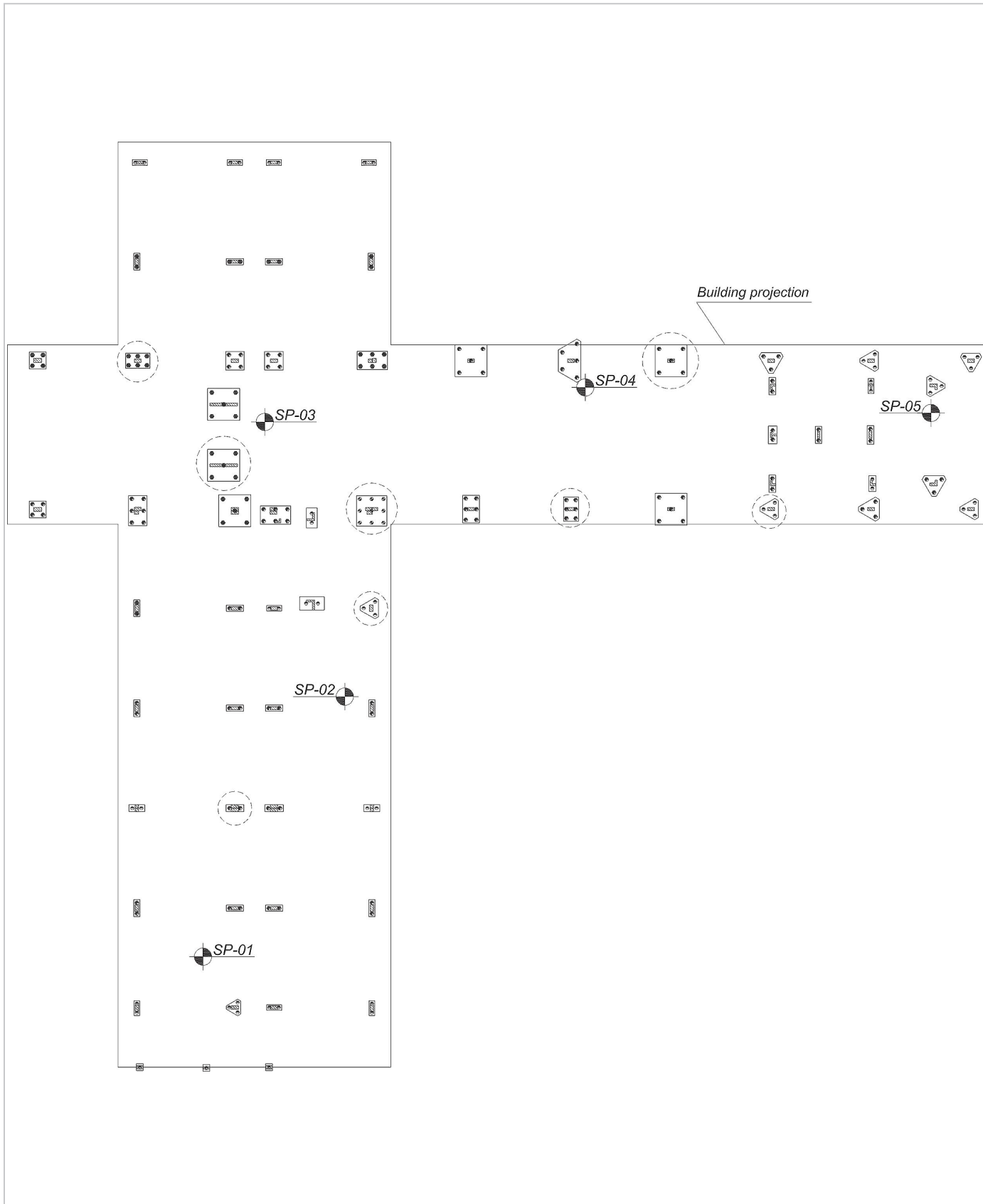


Figure 2 – Floor plan of the multipurpose building foundations.

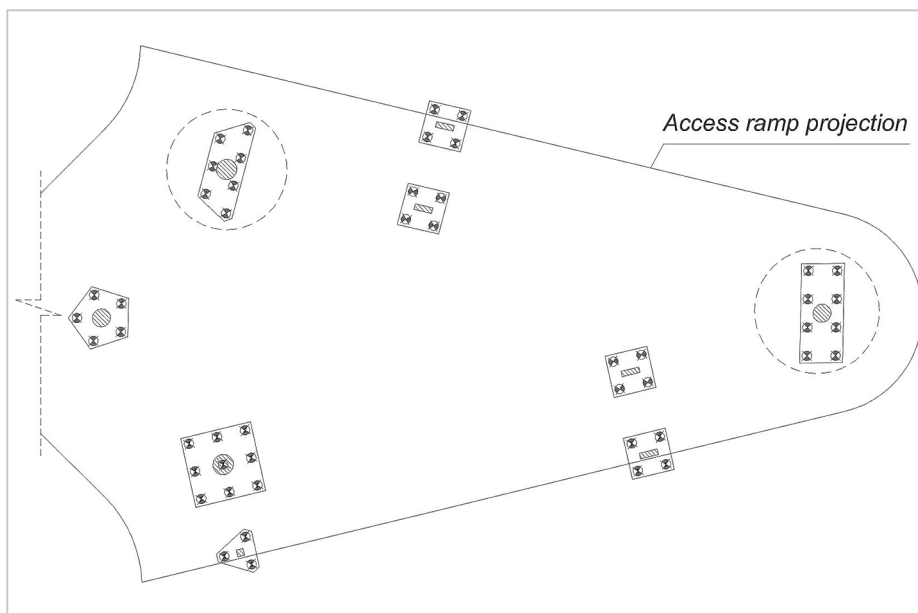


Figure 3 – Floor plan of the access ramp foundations.

3.1 Geological-geotechnical characteristics

The geological-geotechnical characteristics were determined based on five (05) percussion-drilled holes of the SPT type. The results of the five SPT surveys (SP-01 to SP-05) are presented in Figure 4 a). Based on this survey, it was possible to conclude that the soil is homogeneous

and classified as sandy clay with soft to hard consistency, representing the same geotechnical region, according to the provisions of the NBR 6122 (ABNT, 2019). The water level was not found.

The data from the SPT drilling used to determine the geotechnical

load capacity of the piles was indirectly obtained based on geostatistics, using a technique known as ordinary kriging; it is presented in Figure 4 b). For additional details regarding the process of obtaining these data, please refer to Pereira (2020).

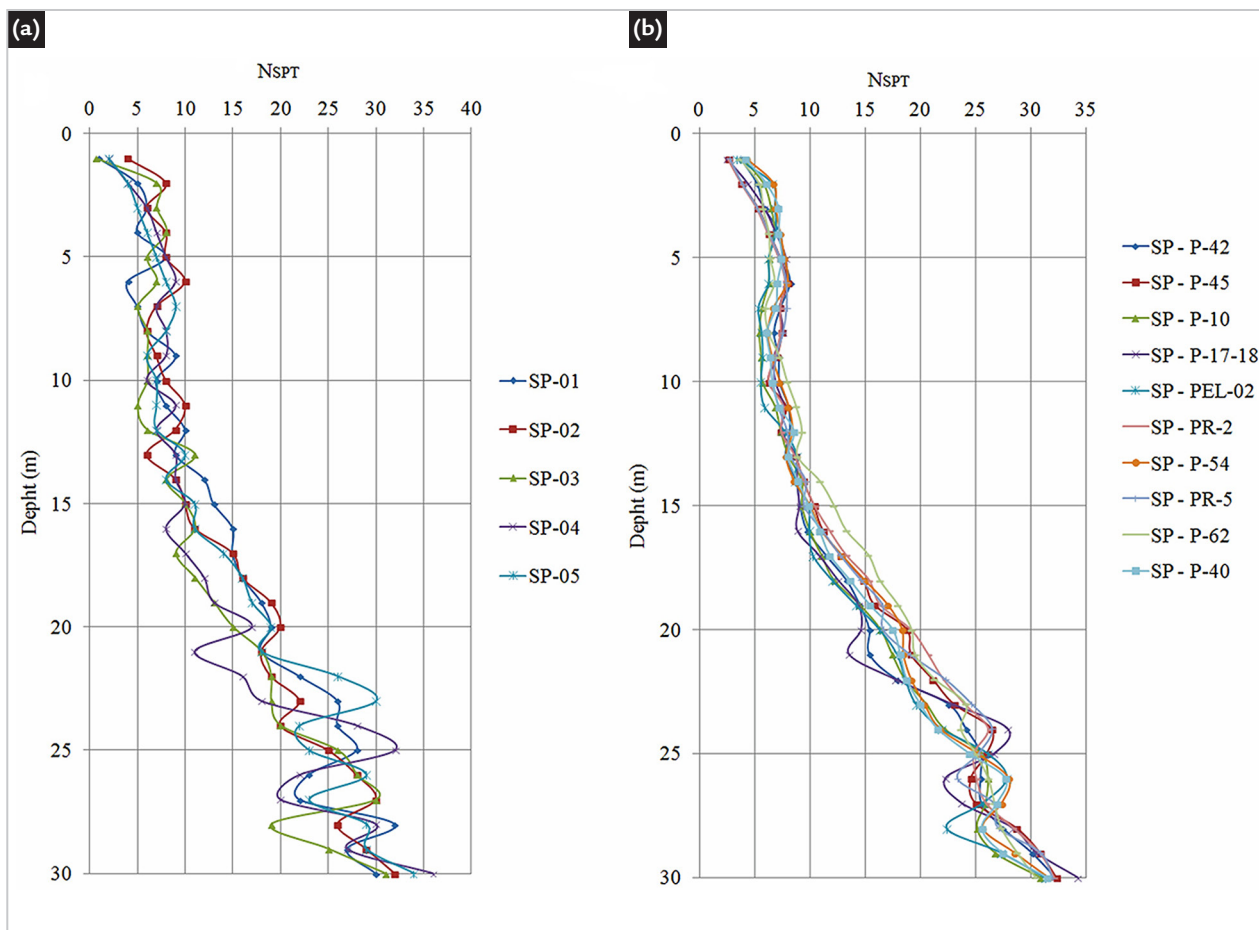


Figure 4 - a) Advance of the original NSPT's in depth (Pereira, 2020);
 b) Representative NSPT's for each pile subjected to DLT obtained via ordinary kriging (Pereira, 2020).

Figure 5 schematically shows the location of the piles subjected to DLT associated with the SPT's profile considered for the use of methodologies for calculating the geotechnical load capacity.

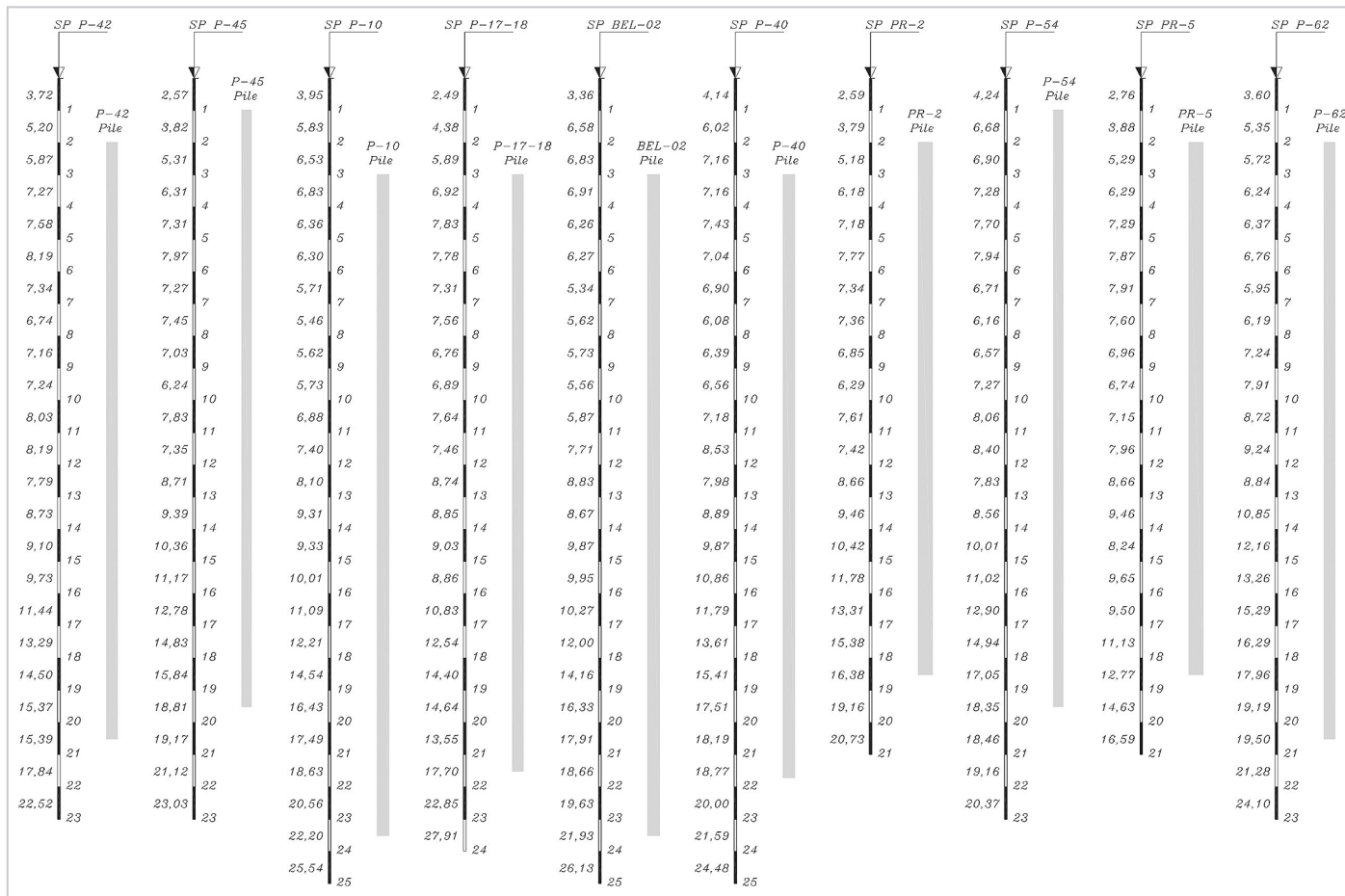


Figure 5 – Location of the piles subjected to DLT associated with the SPT's profile.

3.2 Dynamic load testing

The objective of DLT is to assess the load capacity mobilized by the pile/soil system when a dynamic load is applied. Through a CAPWAP® analysis

(Case Pile Wave Analysis Program), it is possible to separate the resistance mobilized by the pile friction and tip; thus, its geotechnical load capacity can

be obtained. A total of 10 piles that were a part of the project under study were tested, and the results are listed in Table 1.

Table 1 – Results of the CAPWAP® analysis for DLT.

Column/block identification	Type of pile	U	A	L	P _e	R _L	R _p	R
		cm	cm ²	m	kN	kN	kN	kN
P-42	P31	93	624	18.5	900	813	115	928
P-45	P27	81	474	18.5	700	550	124	674
P-10	P34	102	751	20.5	1050	566	335	901
P-17-18	P31	93	624	18.5	900	706	204	910
BEL-02	P34	102	751	20.5	1050	808	448	1256
B-40	P34	102	751	18.7	1050	862	288	1150
PR-2	φ42	132	894*	16.5	1300	640	360	1000
P-54	P27	81	474	18.5	700	451	234	685
PR-5	φ42	132	894*	16.5	1300	546	355	901
P-62	P31	93	624	18.5	900	389	271	660

*Section area filled with concrete, neglecting the hollow area. Where φ is the external pile diameter (cm), U is the perimeter, A is the cross-sectional area, L is the embedded length, P_e is the allowable structural load, R_L is the mobilized shaft resistance, R_p is the mobilized tip resistance, and R is the pile geotechnical load capacity.

4. Results and discussion

Using the SPT survey spectra obtained via ordinary kriging, a representative of each of the piles subjected to DLT and the tip and shaft resistances were calculated. Consequently, the geotechnical load capacities for each of them were determined using the classical semi-empirical methods proposed by Aoki and Velloso (1975), Décourt and Quaresma (1978, 1996), Teixeira

(1996), Pereira (2020). As for the α Method, for each pile, based on the associated SPT profile, the undrained shear strength was calculated based on the average of the correlations proposed by Terzaghi and Peck (1967), Hara *et al.* (1974), Kulhawy and Mayne (1990) and Teixeira and Godoy (1996). On average, the value obtained for the undrained shear strength (s_u) was approximately

103 kPa. To calculate the adhesion coefficient (α), the equation proposed by Kulhawy and Phoon (1993) was used for each pile. The average obtained for the adhesion coefficient was 0.493. These results were compared with the respective resistances mobilized in the DLT using the CAPWAP® methodology, to evaluate the performance of the aforementioned methods.

4.1 Analysis of the resistances mobilized at the piles' tip

Table 2 illustrates the comparison between the tip resistances obtained

via the methods proposed by Aoki and Velloso (1975), Décourt and Quaresma

(1978, 1996), Teixeira (1996), Pereira (2020), α Method and DLT.

Table 2 – Tip resistances (kN) obtained via methods and DLT.

Identification	Aoki and Velloso (1975)	Décourt and Quaresma (1978, 1996)	Teixeira (1996)	Pereira (2020)	α Method	DLT
P-42	286.24	139.15	212.28	222.83	56.29	115.00
P-45	241.91	112.05	178.57	179.44	42.09	124.00
P-10	417.94	205.17	322.73	328.55	71.58	335.00
P-17-18-A	366.62	170.88	236.31	273.63	58.04	204.00
BEL-02	491.93	228.09	355.85	365.25	75.45	448.00
B40-A	376.53	181.32	299.44	290.36	73.46	288.00
PR2-A	422.19	201.22	318.66	322.22	78.18	360.00
P54-A	232.95	106.12	178.71	169.93	43.05	234.00
PR-5-A	366.66	187.10	299.63	299.60	77.74	355.00
P62-A	312.87	149.69	247.45	239.70	59.21	271.00

Figure 6 presents the scatter plot for the tip resistances obtained via each

of the methods and the tested piles as well as their respective adjustment lines

with respect to the reference results, which were obtained via DLT.

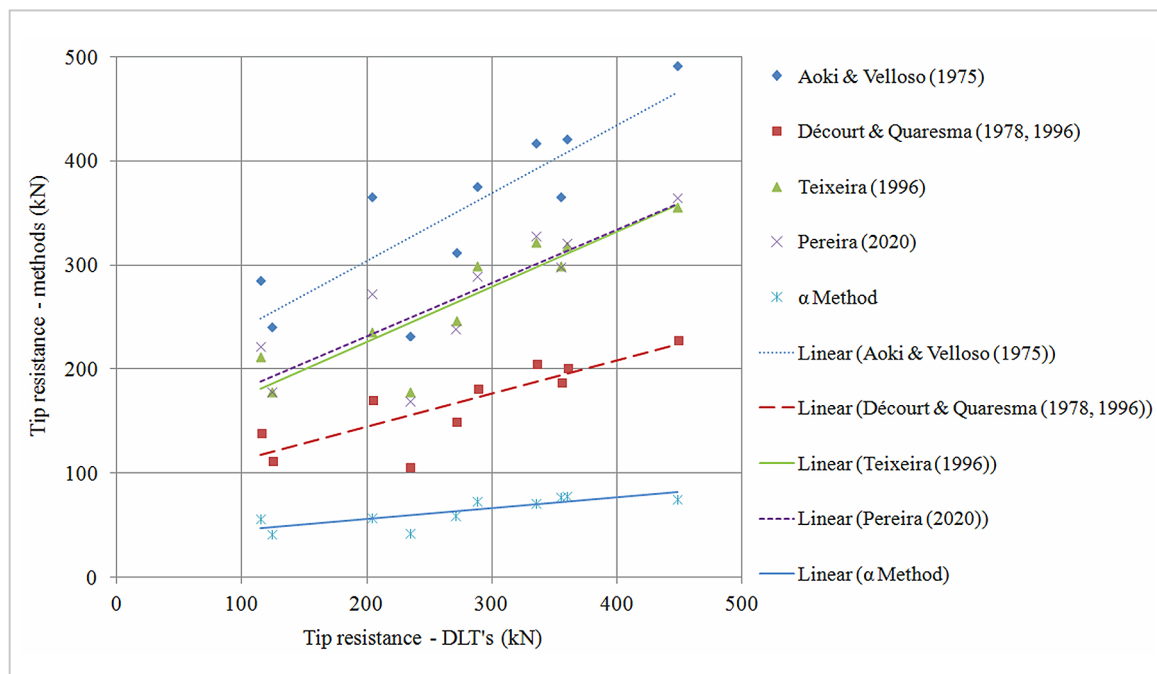


Figure 6 – Dispersion of tip resistances: results obtained via methods x results obtained by DLT.

The adjustment line was defined based on the relationship between the tip resistances obtained via each method for each tested pile and their respective adjustment lines with respect to the reference results. Subsequently, the determination coefficients (R^2) obtained were evaluated for each of the methodologies. The determination coefficients obtained using the semi-empirical methodologies were higher than 0.70; particularly, Teixeira's methodology (1996) yielded an R^2 value slightly exceeding 0.80 and the α Method yielded an R^2 value equal to 0.66. The results according to the methodologies of Bisquerria *et al.*

(2004) and Devore (2006) indicate that there is a strong correlation between the tip resistance results obtained through methodologies and the results obtained through DLT. Hence, it can be said that the tip resistance results obtained using both methods were in agreement with the DLT results. However, the need for discretion is emphasized here because even with this consistency in the tip resistance results, there are scenarios wherein semi-empirical methodologies can yield inaccurate results. Considering the P-42 pile as an example, the reference mobilized tip resistance was 115.00 kN (according to DLT), whereas

the method proposed by Aoki and Velloso (1975) yielded a value of 286.24 kN, which is approximately 149% higher; Teixeira's method yielded a value of 212.28 kN, which is approximately 85% higher; Pereira's method (2020) yielded a value of 222.83 kN, which is approximately 94% higher. Moreover, the method proposed by Décourt and Quaresma (1978, 1996) provides a more measured and coherent result of 139.15 kN, which is 21% higher. The alpha method indicated, in all evaluated cases, to results of tip resistance below the reference values, presenting an average error of 74%.

4.2 Analysis of the shaft resistances mobilized in the piles' shaft

Table 3 illustrates a comparison between the shaft resistances obtained using the methods and DLT.

Table 3 – Shaft resistances (kN) obtained via methods and DLT.

Identification	Aoki and Velloso (1975)	Décourt and Quaresma (1978, 1996)	Teixeira (1996)	Pereira (2020)	α Method	DLT
P-42	499.99	693.17	648.18	583.13	861.24	813.00
P-45	442.89	585.44	554.54	487.43	744.23	550.00
P-10	630.57	871.11	838.47	740.79	1075.89	566.00
P-17-18-A	517.71	706.01	671.14	597.50	874.55	706.00
BEL-02	669.39	911.50	890.09	785.99	1104.64	808.00
B40-A	592.35	817.29	787.66	701.10	994.23	862.00
PR2-A	585.01	844.84	792.13	701.65	1073.46	640.00
P54-A	454.36	599.34	568.89	502.98	752.65	451.00
PR-5-A	581.23	839.35	787.00	695.51	1070.40	546.00
P62-A	529.55	714.77	686.50	607.31	883.32	389.00

Figure 7 depicts a scatter plot of the shaft resistance obtained via each method and for each tested pile as well as their respective adjustment lines with respect to the reference results.

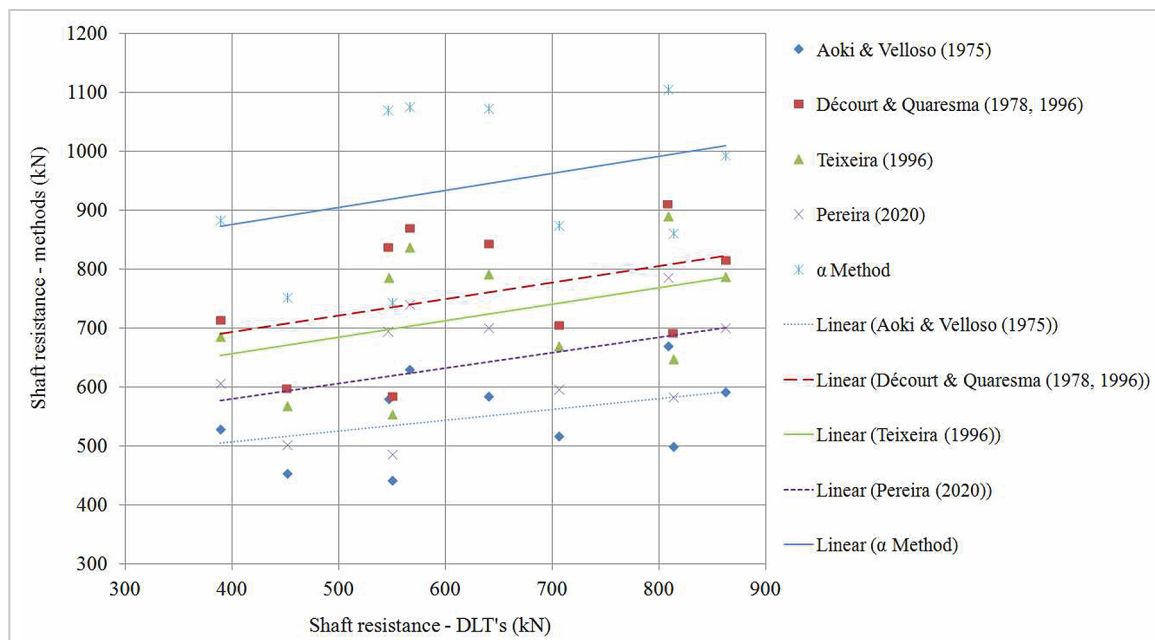


Figure 7 – Dispersion of shaft resistance: results obtained via methods x results obtained via DLT.

The adjustment line was defined based on the relation between the shaft resistances obtained via each method for each tested pile and their respective adjustment lines with respect to the reference results. Subsequently, the values of R^2 obtained for each of the semi-empirical methodologies were evaluated. The determination coefficients obtained through the classical semi-empirical methodologies and the method of Pereira (2020) were in the order of 0.15, whereas the method of Pereira (2020) yielded a

slightly higher value of R^2 , approximately 0.17. The α Method yielded an R^2 value equal to 0.11. According to Bisquerra et al. (2004), such results indicate a low to moderate correlation between data; according to Devore (2006), it is an indicator of a weakly established correlation. Therefore, it can be concluded that both methodologies provided relatively inconsistent results for the prediction of shaft resistance. Moreover, it is suggested that these results should be analyzed in detail; hence, the cases of piles P-45 and P-10,

whose mobilized shaft resistances are 550.00 kN and 566.00 kN, respectively, are considered as examples. In the case of the P-45 pile, Teixeira's method (1996) yielded a value of 554.54 kN, which is higher by less than 1%. However, for the P-10 pile, the same method provided a value of 838.47 kN, which is approximately 48% higher. As for the α Method, it is possible to observe that this method led to an error of only approximately 6% for the P-42 pile and an high error of 127% for the P-62 pile.

4.3 Analysis of the geotechnical load capacities

Table 4 presents a comparison between the geotechnical load capacities

obtained using the methods for the prediction of bearing capacity and DLT.

Table 4 – Load capacities obtained (kN) via methods and DLT.

Identification	Aoki and Velloso (1975)	Décourt and Quaresma (1978, 1996)	Teixeira (1996)	Pereira (2020)	α Method	DLT
P-42	786.23	832.32	860.46	805.96	917.53	928.00
P-45	684.80	697.50	733.11	666.86	903.33	674.00
P-10	1048.51	1076.28	1161.20	1069.34	932.82	901.00
P-17-18-A	884.33	876.89	907.45	871.13	919.28	910.00
BEL-02	1161.32	1139.60	1245.94	1151.24	936.69	1256.00
B40-A	968.88	998.61	1087.09	991.46	934.70	1150.00
PR2-A	1007.20	1046.06	1110.78	1023.88	939.42	1000.00
P54-A	687.30	705.46	747.60	672.92	904.29	685.00
PR-5-A	947.89	1026.44	1086.63	995.11	938.98	901.00
P62-A	842.43	864.46	933.94	847.00	920.45	660.00

Figure 8 presents the scatter plot of the geotechnical load capacities obtained

using each method for each tested pile and their respective adjustment lines with

respect to the reference results.

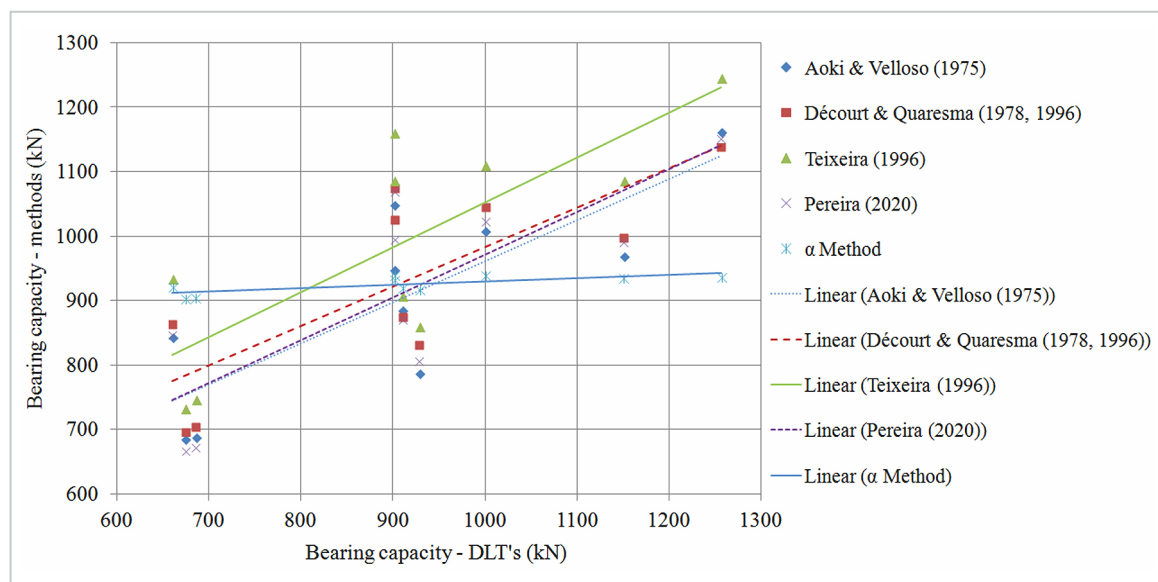


Figure 8 – Dispersion of the geotechnical load capacity: results obtained via methods x results obtained via DLT.

The adjustment line was defined based on the relationship between the geotechnical load capacities obtained

via each of the methods for each tested pile and their respective adjustment lines with respect to the reference results. Sub-

sequently, the values of R^2 obtained for each of the methodologies were evaluated. The determination coefficients obtained

through the semi-empirical methodologies were in the order of 0.63 (average value) and the α Method yielded an R^2

value equal to 0.55. According to Bisquerra *et al.* (2004), these results indicate a strong to a significantly strong correla-

tion between data; according to Devore (2006), it is an indicator of a moderate to a strong correlation.

4.4 Performance analysis of semi-empirical methods

Due to the different dimensions of the piles constituting the project, the values of the maximum resistances obtained using DLT were divided by their cross-sectional areas to eliminate the cross-section variable. The average

value obtained was 13400.15 kPa, and the standard deviation was 2259.01 kPa, configuring a variation coefficient of the order of 17%. For unit compatibility, the same process was performed for the load capacities obtained via the semi-empirical/

theoretical methodologies. The data is consolidated in Table 5. It should be noted that for the reasons presented, the results of the geotechnical load capacity or ultimate mobilized resistance presented in Table 5 are in kPa.

Table 5 – Geotechnical load capacities or ultimate mobilized resistances (kPa).

Identification	Aoki and Velloso (1975)	Décourt and Quaresma (1978, 1996)	Teixeira (1996)	Pereira (2020)	α Method	DLT	DLT -17%*	DLT +17%**
P-42	12599.90	13338.50	13789.43	12916.03	14704.01	14871.79	12343.59	17400.00
P-45	14447.26	14715.09	15466.55	14068.78	19057.59	14219.41	11802.11	16636.71
P-10	13961.56	14331.34	15462.03	14238.88	12421.04	11997.34	9957.79	14036.88
P-17-18-A	14172.00	14052.67	14542.45	13960.42	14732.05	14583.33	12104.17	17062.50
BEL-02	15463.64	15174.37	16590.35	15329.43	12472.57	16724.37	13881.23	19567.51
B40-A	12901.19	13297.03	14475.29	13201.86	12446.07	15312.92	12709.72	17916.11
PR2-A	11266.27	11700.88	12424.87	11452.80	10508.05	11185.68	9284.12	13087.25
P54-A	14500.11	14883.20	15772.18	14196.62	19077.85	14451.48	11994.73	16908.23
PR-5-A	10602.79	11481.47	12154.74	11130.98	10503.13	10078.30	8364.99	11791.61
P62-A	13500.47	13853.48	14967.06	13573.72	14750.80	10576.92	8778.85	12375.00

*Results 17% lower than those obtained via DLT.

** Results 17% higher than those obtained via DLT.

To analyze the performance of the semi-empirical methods, the methodology presented by Velloso (2019) was applied. As a result, the “degree of reliability” was defined as the amount or percentage

of points located between the deviation lines of more than 17% and less than 17%, with respect to the ultimate load mobilized in the DLT, as depicted in Figure 9. It is indicated that the considered

deviation of approximately $\pm 17\%$ refers to the variability observed in the ultimate resistances mobilized in the DLT, for the 10 tested piles that are in the same representative region.

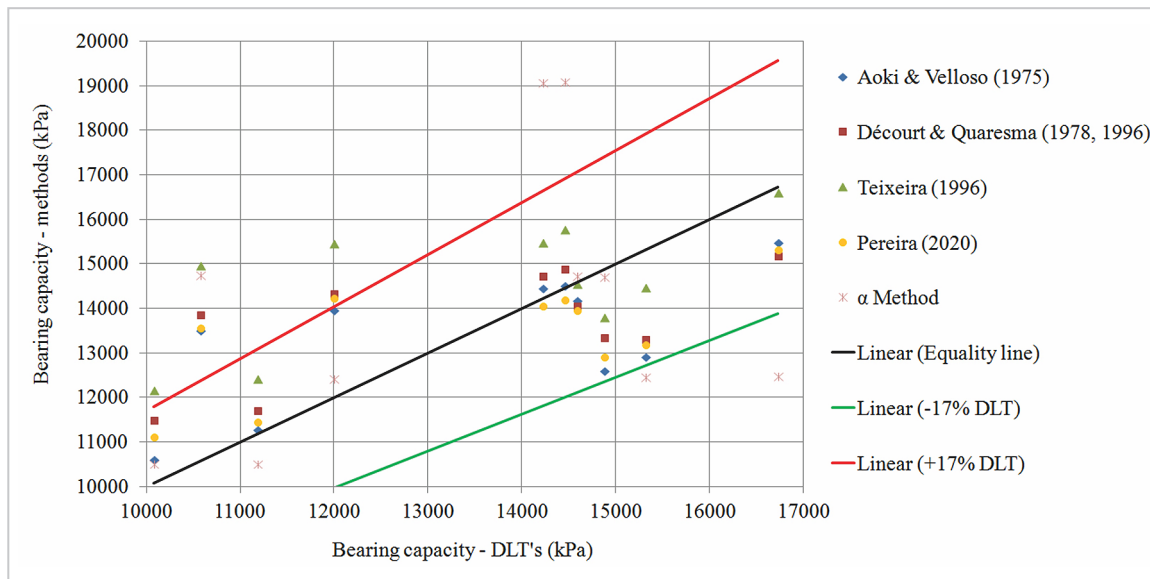


Figure 9 – Performance analysis of semi-empirical/theoretical methods.

As shown in Figure 9, the coordinated pairs were plotted on a graph represented by “Load capacity by DLT v/s Load capacity by semi-empirical meth-

ods.” The graph illustrates the equality line that corresponds to the desirable value (values calculated using the semi-empirical method and equal to those obtained via

DLT) and two lines corresponding to deviations of + 17% and -17% in relation to the equality line, delimiting an acceptance range termed as the “hit zone.”

From Figure 9, it is evident that a majority of the points are located in the region considered as the “hit zone,” qualitatively indicating the robust performance of the semi-empirical methods used to determine the geotechnical load capacities of the piles. The best performance was achieved by the method proposed by Aoki and Veloso (1975), which presented a single point beyond the “hit

zone,” i.e., 90% of the calculated results were in the expected region. This was followed by the method of Décourt and Quaresma (1978, 1996) and the method of Pereira (2020), which yielded 80% of the results in the “hit zone”. The method of Teixeira (1996) yielded 70% of the results in the expected region. Finally, the α Method presented 50% of the results in the “hit zone.”

It should be noted that the results are presented in terms of load capacity. Therefore, if a minimum safety factor of two (2) is applied, according to the NBR 6122 (ABNT, 2019), to the results beyond the “hit zone” the permissible load will be substantially lower than the ultimate resistance, thereby indicating a certain security of the values calculated, from a deterministic point of view.

4.5 Discussion about the methods for calculating the geotechnical load capacity of piles

Several researchers have presented comparative studies between the results of geotechnical load capacity obtained via semi-empirical/theoretical methods and those obtained via DLT and/or static load testing (Cabette, 2014; Alledi *et al.*, 2015; Veloso, 2019). Each of these studies have reported considerably different conclusions regarding the observed accuracy of these methods as compared with static load tests and/or DLT. It is inferred that this observed difference in the performance is a result of factors such as the pile characteristics in terms of typology and geometry, geological-geotechnical conditions, variability and lack of criteria for the definition of geotechnical representative regions, issues inherent to the executive quality of field tests and their interpretations, and particularities of the methods. It is known that the geological-geotechnical formation has a significant influence on pile resistance (Veloso, 2019). The particularities of semi-empirical methods are specifically highlighted herein, which have already been presented by several authors, such as Schnaid and Odebrecht (2012). Therefore, the validity of such methods is limited to the regional constructive

practices and the specific conditions of historical cases used in their development.

Semi-empirical methods have been widely used for the prediction of geotechnical load capacity of the pile. This significant use technically culminates in the development of new methods and comparative studies between methods and analyses of their performance, as reported in studies such as Powell *et al.* (2001), Momeni *et al.* (2013), and Benali *et al.* (2018).

Amann (2010) states that semi-empirical methods are not universal and should only be applied with reservations to soils from different locations than those originally researched by the researchers proposing the methods. The normative provisions of NBR 6122 (ABNT, 2019, p. 22) regarding the semi-empirical methods are included, which states that “[...] the validity domains of their applications must be observed, as well as the data dispersions and the regional limitations associated to each method”.

Theoretical methods for the estimation of pile load capacity are not used as often in the practice of Brazilian Foundation Engineering since, as previously evi-

denced, as a rule, the SPT is the most used survey, and in most massive projects foundations, it is the only available test, which makes the use of semi-empirical methods that directly correlate SPT measurements with the load capacity of the piles prevail. In the absence of test results for determining soil resistance parameters (cohesion and/or friction angle), these parameters will need to be estimated by correlation for the application of theoretical methods, which may result in another source of errors for the mathematical model.

Therefore, it is suggested that an estimation of the geotechnical load capacity of the piles be carried out using several relevant semi-empirical/theoretical methods, especially with respect to the type of pile and the region where the project is carried out. Additionally, the stresses mobilized along the shaft and the tip of the piles should be calibrated based on static load tests and/or DLT. Moreover, regionalized methods should be developed, and a database of these parameters should be created, to ensure that such data can be subjected to statistical analyses and consequently improved, as proposed by Pereira (2020).

5. Conclusions

In the context of geotechnical/foundation engineering, it can be stated that both the semi-empirical methodologies yielded relatively consistent results with relative accuracy, for the prediction of the geotechnical load capacity with respect to this case study. Despite the robust results obtained in the determination of the load capacity, it is necessary to exercise caution when using semi-empirical methodologies. Although a trend of consistent results in terms of the tip resistance was obtained, there are certain scenarios wherein such semi-empirical methodologies can provide inaccurate results, as demonstrated herein, where tip resistances approximately 149%

higher than the reference value were obtained. Similarly, values ranging from less than 1% and more than 48% of the reference value were obtained using the same method, when determining the shaft resistances. Regarding the performance analysis, it was observed that a majority of the points were found in the region known as the “hit zone”, thereby indicating the robust performance of the semi-empirical methods used to forecast the geotechnical load capacity of the piles. As for the theoretical method known as α Method, it was possible to observe a lower performance than the semi-empirical methods considered, since it obtained only 50% of the results

in the “hit zone”.

Finally, based on reviewed literature and the performed analyses, it is possible to conclude that all semi-empirical/theoretical mathematical models for the prediction of geotechnical load capacity of piles are subjected to restrictions of applicability and definitiveness. This is because the method is elucidated considering regional resistance conditions and the result of the history of the local geological/geotechnical formation. Even the methods already enshrined in Foundation Engineering practice should be limited to the locations whose test results are available for the calibration of the mathematical model.

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