

Prediction of Bearing Capacity of Reaction Piles in a Collapsible Soil by In Situ Tests

Previsão da Capacidade de Carga de Estacas de Reforço em Solos Colapsíveis por Ensaios de Campo

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ABSTRACT: There are several profiles of unsaturated collapsible soils in tropical climate regions. These soils show changes in their mechanical behavior due to variations in soil suction. Collapsible soils are not always identified during the site characterization for foundation design, nor is the variability effect assessed, which can lead to pathologies as differential settlement in foundations. Reaction piles, or Mega piles, are currently used as foundation reinforcement, with operators executing them based on their own experience rather than using a rational approach. The pile load test is the best way to predict the foundation behavior, but it is expensive and time-consuming. An alternative is the use of semiempirical methods based on in situ tests. The Dynamic Light Penetrometer (DPL) is an interesting test, as it is quick, simple to perform and easily accessible even in sites with difficult access. This paper evaluated the prediction of bearing capacity for reaction piles based on DPL and SPT and pile load tests carried out on a variable collapsible soil profile and assessed the applicability of the DPL for foundation reinforcement design using a semiempirical approach, considering the effects of inherent variability of soils in the in situ test data.

KEYWORDS: DPL, SPT, bearing capacity, foundation reinforcement, site variability.

1 INTRODUCTION

Site investigation is an extremely important stage in geotechnical design. It enables definition of the site stratigraphic, the ground water table position, as well as an estimate of the soil parameters and the identification of the problematic layers.

The use of in situ tests allows the site profile to be idealized and the mechanical parameters to be estimated for the study site (Giacheti & Queiroz 2004). In situ testing also allows the verification of behavioral variations that may occur throughout the depth, both due to the inherent soil variability, geometry of the layers, the geological formation, the history of transformations; and due to seasonality, which influences soils formed in regions with a tropical climate. Therefore, it is possible to reduce the uncertainty associated with soil parameters estimative by a better site characterization through a more complete investigation.

Brazilian soils are characterized by wetting and drying cycles, leading to the formation of unsaturated profiles up to great depths (Fernandes 2022). The interactions of the liquid, gaseous and solid phases give unsaturated soils the action of the suction component, subject to variations resulting from the seasonality of the dry and rainy seasons, leading to variations in the geotechnical parameters (Dos Santos 2003, Mota 2003, Lessa 2005, Saab 2016, Rocha 2018). This factor, however, is often overlooked in early design and site investigation stages. Geotechnical projects with satisfactory performance and safety should take into account the variability of the site. A frequent characteristic of tropical

unsaturated porous soils is collapsibility. Collapsible soils can show a significant decrease in volume associated with an increase in the degree of saturation without a significant increase in the external overloads applied (Vilar & Rodrigues 2011). Rocha et al. (2023) point out that a large part of the Brazilian territory is covered by collapsible soils, especially in the Center-South and Northeast regions of the country.

Collapsible soils have a metastable structure characterized by the existence of a temporary resistance generated by suction and, in some cases, by water-soluble cementing elements (Saab 2016). This structure, however, can be easily broken by external agents, configuring the mechanism of collapse, with flooding reducing the suction of the soil, which eliminates the elements that maintain this structure (Saab et al. 2023).

Collapsible soils are considered problematic geomaterials. This problem is even more acute in residential foundation projects, which rarely have a proper design or site investigation, and differential settlements can occur. A common practice to solve these problems is the use of reaction piles or Mega piles to reinforce foundations.

Golombek (1996) and Mantovani et al. (2020) points out that the number of publications involving Mega piles is quite small although these piles are widely used. This is probably because problems in buildings are usually caused by design or execution errors, which makes publications on the use of reinforcements uncomfortable. There is rarely an executive project in place, with operators executing them based on their own experience rather than

a rational approach. The lack of information on the subject creates distance and divergence in concepts between designers and executors of Mega piles, thus hindering proper scientific, professional, and ethical performance in the execution of this type of pile (Donadon 2009). The difficulty associated with carrying out pile load tests to verify the behavior of a foundation element makes it even more difficult to obtain information about these piles.

Semiempirical methods are used to predict pile bearing capacity, based on in situ tests data to overcome this difficulty. These techniques are widespread, and the results obtained are the basis for estimating geotechnical design parameters in Brazil (Schnaid & Odebrecht 2012). However, despite being widespread, this tests present factors that influence their routine, whether due to non-compliance with SPT standards or practical lack of experience with CPT and DMT tests in Brazilian practice.

An alternative to overcome such difficulties is the use of the Dynamic Penetration Light (DPL) test, an almost continuous penetration test with no sample collection, which aims to estimate soil resistance and deformation parameters, as well as defining the soil profile. The test is similar in practice to the SPT, with the test consisting of advancing a conical tip 0.10 m into the soil, measuring the number of blows for advancing this probe, resulting in an index called N_{10} , which can be used in bearing capacity predictions. A major advantage of the test is its accessibility, since it can be carried out in areas that are difficult to reach due to its compact configuration (Cunha & Nilsson 2003). It is simple to operate and allows continuous recording of subsoil data, even if there is little operational space (Bastos 2016).

This paper presents an initial analysis of the applicability of the DPL test in predicting the bearing capacity of Mega piles in collapsible soils based on semiempirical methods currently used in Brazilian practice, such as Aoki & Velloso (1975) and Décourt & Quaresma (1978). SPT data are also used in predicting the bearing capacity of these piles. The aim is also to understand the effects of a better site characterization on the foundation design, assessing the inherent variability to reduce uncertainties with a greater number of in situ tests.

2 STUDY SITE

The interpreted DPL tests and pile load tests (Boni et al. 2006, Sitta 2008, Donadon 2009) were carried out at the Unesp Experimental Research Site in the city of Bauru, state of São Paulo, Brazil, close to the Meteorological Research Institute (IPMet) (Figure 1). The pile load tests, as described by the authors, were carried out in real magnitude piles by applying loads using a jack, mechanical extensometers, and a reaction system, making it possible to obtain reliable parameters for foundation reinforcement projects.

The city of Bauru, state of São Paulo, Brazil, is characterized by the collapsibility of its soil (Saab 2016, Rocha 2018), that is why the study of Mega piles is interesting for civil engineering works in the region. Boni et al. (2006), Sitta (2008) and Donadon (2009) carried out load tests on 8 m long Mega piles in natural ground conditions and found great variability in the results obtained. Donadon (2009) also studied three 6 m long piles under natural ground conditions, and again found great variability in the results. This variability, inherent to the site, must be considered, since a proper site characterization provides a better understanding of the test data as well as the behavior of the local soil (Giacon Junior et al. 2019).

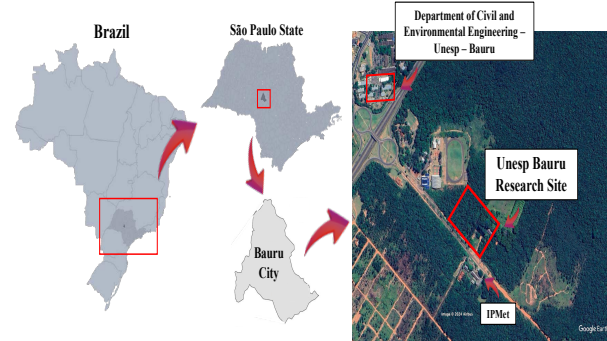


Figure 1. Study site location.

The site is geologically characterized by Sandstone rocks of the Bauru Group. The soil is characterized as a fine to medium sand, slightly clayey, of colluvial origin up to 13 m deep, formed by weathering typical of the tropical region. The colluvium soil is over the residual soil from the Bauru Sandstone, with a texture very similar to that of colluvial soil (De Mio 2005). It is an unsaturated porous soil with a high saturated hydraulic conductivity, which means that its engineering properties and parameters are easily affected by rainwater and transient groundwater changes in a way that is difficult to predict (Fernandes 2022).

The collapsible behavior of this soil was verified by Lobo (1991) in studies carried out on short piles cast in place. The author found that flooding the soil prior to testing resulted in a loss of bearing capacity of around 30 to 50%. Saab et al. (2023) point out that the collapsible behavior at the design stage can be a problem if it is not properly considered.

2.1 Particularities of the Study Site

The Unesp Experimental Research Site has been studied by several authors. Ferreira et al. (1996) and Donadon (2009) point out that this area is not quite representative of Bauru region, with some sections showing a very different behavior in terms of soil resistance. This aspect, however, can be assessed as the result of the geotechnical variability of the soil site, an inherent characteristic of the material that is complex to deal with, but which can be eliminated with a proper site characterization, as discussed by Phoon & Kulhawy (1999) and Giaccon Junior et al. (2019). De Mio (2005) pointed out about the variability of this study site due to the geological formation process in this area.

Giaccon Junior et al. (2019) also point out that the predicting the load bearing capacity of piles in this soil type in an area with lower variability, the dispersion of the results needs at least ten tests to be representative. So, a proper site characterization is very important for understanding the behavior of the site and predicting the load bearing capacity of the foundation elements.

So, the Unesp Experimental Research Site located close to the IPMet can be divided into two areas: the "Area A", which shows a behavior considered "unrepresentative" by Ferreira et al. (1996) and Donadon (2009), and the "Area B", considered representative of soil the city of Bauru. The boundaries between these two areas are not well defined geometrically. Several SPTs were conducted on these two areas and they were used to identify these different areas and to correlate them with the DPL tests. A total of five SPTs were carried out in the Area A and six SPTs in the Area B. The

location of all these tests, as well as the description of this experimental site, is presented in detail by Ferreira et al. (1996), Giacheti (2001) and Donadon (2009).

3 IN SITU TESTS

3.1 DPL

The DPL is a quasi-continuous penetration test, without collecting soil samples. The test data are presented in terms of soil resistance and deformation parameters, as well as an estimate of the site profile, based on the changes observed in a penetration index.

The DPL test is carried out in the same way as the SPT, applying blows to advance a conical tip with an angle between 60° and 90° into the ground, resulting in a resistance index, N_{10} , which corresponds to the advance of 0.10 m of the probe into the ground. It is simple to operate and has advantages due to its compact configuration and accessibility, allowing continuous recording in areas that are difficult to access (Cunha & Nilsson 2003, Bastos 2016, Almeida 2023).

The DPL used was the “Barra Mina” type with manual hammer lift and drive, manufactured by SOLOTEST in accordance with the specifications of ISO 22476-2. The tests followed the recommendations of ISO 22476-2 and DIN 4094.

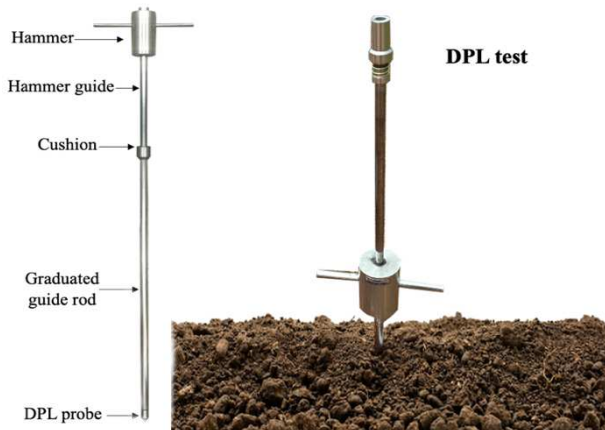


Figure 2. Schematic representation of the equipment.

Seven tests were carried out to a depth of 8 m or if a stop criterion was reached, i.e. when 50 blows were applied three times in a row.

The tests were at least 1 m apart from the SPT, to incorporate the results into the study of the behavior of common piles used to reinforce small to medium-sized buildings. Three tests were carried out in the Area A and four tests in the Area B.

3.1.1 Correlations between DPL and SPT

Saraiva Junior and Garcia (2023) point out that the energy levels between DPL and SPT differ significantly. The SPT delivers up to ten times more energy than the DPL, considering only the contribution of the hammer.

Several authors have tried to establish correlations between the resistance indexes pertaining to the tests (N_{10} and N_{SPT}). One of the correlations used by authors such as Sanchez et al. (2011), Aguiar

et al. (2018) and Saraiva Junior and Garcia (2023) is that shown in Equation 1.

$$N_{10} = a + N_{SPT} b \quad (1)$$

The coefficients a and b are relative to the test site and its geological characteristics.

Sanchez et al. (2011) proposed a way of correlating the N_{10} and N_{SPT} indexes using linear correlations, as shown in Equation 1. The author inserted the N_{10} parameter in three different ways in the correlation: (i) the average of all the N_{10} parameters obtained at each meter depth; (ii) the average of the N_{10} values obtained in the last 0.30 m of each meter depth; (iii) the sum of the N_{10} values and those obtained during penetration of the last 0.30 m of the stem at each meter depth. Aguiar et al. (2018) proposed another way: (iv) average of the N_{10} parameters in the 0.30 m of the SPT zone of influence.

3.2 Pile Bearing Capacity and Semiempirical Prediction Methods

The bearing capacity (Q_U) of a pile is the capacity at which settlement continues to occur without any further increase in load (Lobo 1991). The most appropriate way to define it is by carrying out a load test on the pile inserted into the soil at the site of interest. On construction sites, it is pertinent to carry out these tests at the locations where the foundations will be built.

Load tests require a large structure, making them expensive, time-consuming and difficult to carry out. These characteristics make it unfeasible to carry out these tests to assess site variability. Therefore, the results of load tests on construction sites rarely consider the site variability.

Methods based on the results of in situ tests to predict the load bearing capacity of piles are commonly used to overcome these difficulties.

Saraiva Junior and Garcia (2023) highlight the use of DPL in predicting the load bearing capacity of piles. The authors argue that the test allows for better discretization of the site profile and propose ways for using the N_{10} index in semiempirical methods. Some of the usual methods for predicting pile load capacity are presented below.

3.2.1 Aoki & Velloso Method (1975)

The Aoki & Velloso method (1975) uses the tip resistance and the lateral friction to estimate the bearing capacity of a pile. It was initially developed using CPT data, being further adapted for the SPT.

This method considers the type of soil, using coefficients such as K and α , as well as the geometric and constitutive parameters of the pile, such as its perimeter, the cross-sectional area of the tip, and the type of pile. The resistance R is given by:

$$R = \frac{K N_P A_P}{F_1} + \frac{U}{F_2} \sum (\alpha K N_L \Delta L) \quad (2)$$

Wherein A_P is the area of the pile tip; U is the pile perimeter; ΔL is the thickness of the soil layer; N_L is the average N_{SPT} for each soil layer; N_P is the N_{SPT} at the pile bearing elevation; K and α are soil coefficients; F_1 and F_2 are correction factors for the tip and lateral resistances, respectively.

3.2.2 Décourt & Quaresma Method (1978)

The Décourt & Quaresma method (1978) was proposed based on SPT data. The method was later updated by Décourt (1996) and the resistance is given by:

$$R = \alpha C N_p A_p + \beta U 10 \left(\frac{N_L}{3} + 1 \right) \Delta L \quad (3)$$

Wherein A_p is the area of the tip of the pile; U is the perimeter of the pile; ΔL is the thickness of the soil layer; N_L is the average N_{SPT} along the shaft; N_p is the N_{SPT} at the tip or base of the pile, obtained from three values: the one corresponding to the level of the tip or base, the one immediately before it, and the one immediately after it; α and β are coefficients depending on the type of pile and the type of soil; C is the coefficient characteristic of the soil. It is worth noting that the N_L calculation does not consider the values that will be used to assess the tip resistance.

3.3 Mega Pile Load Test

The reference pile load tests for this study were those carried out by Boni et al. (2006), Sitta (2008) and Donadon (2009) at the study site. The concrete Mega piles have a diameter of 0.20 m and lengths of 6 and 8 m. The tested piles are those commonly used in foundation reinforcement projects.

A total of three load tests under natural conditions for the 6 m length piles and six load tests under natural conditions for the 8 m piles were analyzed. This study site shows great geotechnical variability, as discussed by Ferreira et al. (1996) and Donadon (2009), with pile load tests spaced 10 m apart showing a great difference in load bearing capacity.

Figure 3 shows the characteristic variable behavior observed for the 6 m long piles. As it is common in foundation engineering practice, the tests were executed in a short period of time and in similar water content conditions. Thus, the seasonal influence on the results can be eliminated (Réthati 1988, Phoon & Kulhawy 1999). It reinforces the hypothesis of the geotechnical variability of the site, which could be eliminated with better characterization of the site, as presented by Giacon Junior et al. (2019).

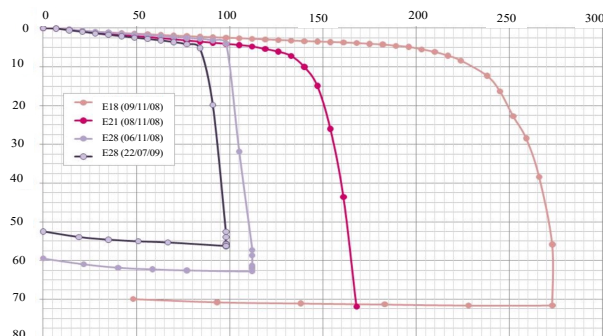


Figure 3. Load versus displacement curves for 6 m length Mega piles for the natural condition (adapted from Donadon 2009).

3.3.1 Mega Pile Load Test Interpretation

The used criterion to interpret the pile load test was the NBR 6122 (2022). According to that standard, the ultimate load corresponds to the settlement obtained in Equation 4.

$$\Delta_r = \frac{P_r L}{AE} + \frac{D}{30} \quad (4)$$

Wherein Δ_r is the conventional failure settlement; P_r is the conventional failure load; L is the length of the pile; A is the cross-sectional area of the pile; E is the modulus of elasticity of the pile material; and D is the diameter of the circle circumscribed by the pile.

The settlement is calculated from an arbitrary value. To achieve this, a straight line is plotted which crosses the axis of settlement at $D/30$. Figure 4 shows an illustration of the proposed method.

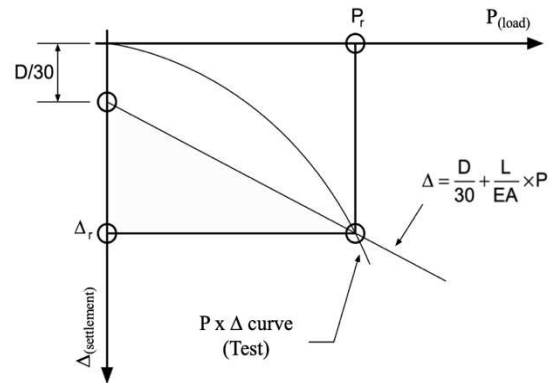


Figure 4. Pile load test interpretation criterion (adapted from ABNT NBR 6122).

The load bearing capacity of the Mega piles considered in this study was determined according to the NBR 6122 (2022) method based on the interpretation of the load versus displacement curves, and the values are shown in Table 1.

Table 1. Considered Mega piles bearing capacity.

Length (m)	Ground condition	Ultimate bearing capacity (kN) NBR 6122	Pile	Test campaign
6	Natural	98.9	E28	Donadon (2009)
6	Natural	134.5	E21	
6	Natural	224.7	E18	
8	Natural	149.6	E26	
8	Natural	156.7	E22	
8	Natural	194.8	E20	
8	Natural	230.4	E23	Boni et al. (2006)
8	Natural	212.0	E1	
8	Natural	180.0	E3	

Considering the results shown in Table 1, it is possible to notice the site variability effect in pile load tests when comparing the ultimate bearing capacity of a pile to the other with the same length. Figure 5 shows the variation of the ultimate bearing capacity for each pile length when comparing the in situ pile load tests. The 6 m long piles vary from a 0.44 to 2.27 range, and the 8 m long piles vary from a 0.65 to 1.54 range.

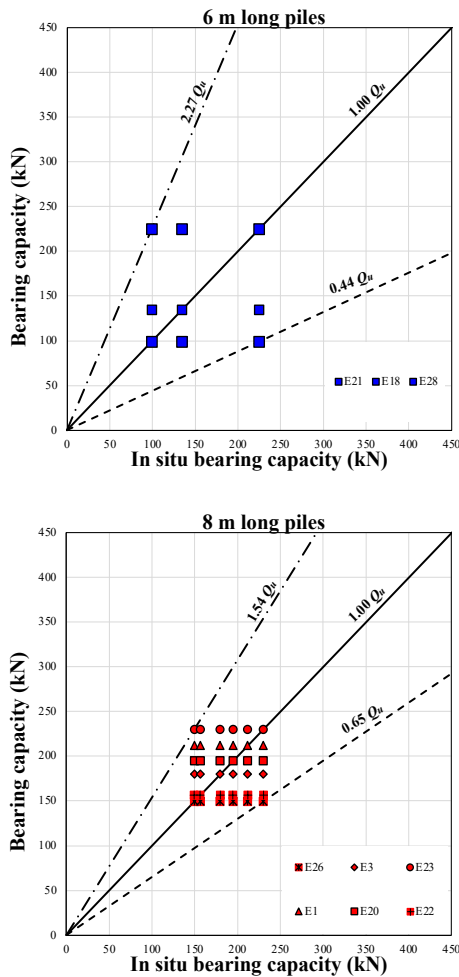


Figure 5. Variation in the bearing capacity of the 6 and 8 m long piles observed in the load tests performed at the study site.

4 TEST DATA AND ANALYSES

4.1 In Situ Tests

The DPL tests were carried out in the Area A and in the Area B to compare them with the SPTs available on the sites. All the in situ test data were combined in the analysis of the study site to incorporate the geotechnical site variability.

4.1.1 Area A

Figure 6 shows the DPL test results for the Area A, with penetration resistance profiles, N_{10} , as well as the mean values ($\bar{N}_{10,A}$), ranges of variation from the standard deviations (σ) and coefficients of variation (CV) for the measurements.

The results indicate a large variation in the resistance profiles for the first few meters, with the highest coefficient of variation values. This happens since the topsoil is more subject to compaction as well as the presence of roots that may influence the resistance of this horizon.

The resistance increases with depth and the variation decreases, tending towards a minimum level, which can be seen in the low standard deviation and coefficient of variation values.

Figure 7 shows the SPT results, with averages ($\bar{N}_{SPT,A}$) and standard deviation, as well as the coefficient of variation (CV). It is possible to see a similar behavior, with a marked increase in resistance, except for test “N10”, which showed a slight decrease in the at 8 m depth.

4.1.2 Area B

Figure 8 displays the DPL test results for the Area B, showing the penetration resistance profiles, N_{10} , with the mean values ($\bar{N}_{10,B}$), standard deviations (σ) and coefficients of variation (CV) for the measurements.

The results indicate a large variation in the resistance profiles for the first few meters, with the highest coefficient of variation values, as in the previous area.

The resistance increases with depth and the variation decreases, which can be seen in the low standard deviation and coefficient of variation values.

However, the increase in resistance was not as pronounced for this area as the previous one. This is an indication of the variability of the terrain.

Figure 9 shows the SPT results, with averages ($\bar{N}_{SPT,B}$) and standard deviation, as well as the coefficient of variation. The behavior is similar, with a coefficient of variation in the 25 to 30% range.

4.1.3 Combining Area A and Area B

The DPL and SPT data for both areas show the same general trend with the resistance increasing along the depth. It is possible to see the great variability at the studied site when comparing the data for both areas.

Analyzing the of the pile load tests, especially those from Donadon (2009), the large discrepancy may be associated with the site variability, as well as in the site characterization that is not sufficient to be considered representative.

An average value was assumed for the site to incorporate the variable of geotechnical variability. It was done to better understand the behavior of the soil, and thus help understand the load *versus* displacement curves and advance their interpretation when considering a site with a heterogeneous soil.

Figure 10 shows the average results for the study site, in terms of penetration resistance profiles, average \bar{N}_{10} compared to the other areas, as well as the standard deviation values (σ) and coefficients of variation (CV) for the measurements. The coefficient of variation is high, tending towards 50% throughout all the profile.

The same was done for the SPT shown in Figure 11. When comparing the SPT data, the profile of the coefficient of variation tends towards values above 50% when analyzing an average value for the site.

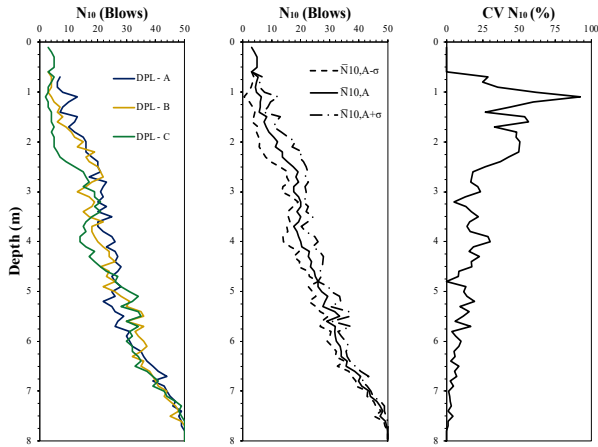


Figure 6. DPL test profiles for the Area A.

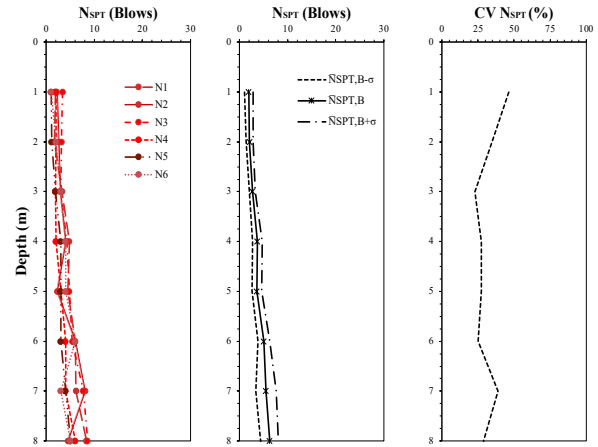


Figure 9. SPT profiles for the Area B.

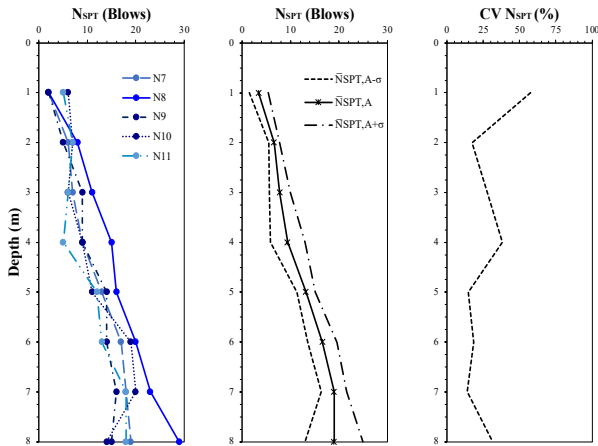


Figure 7. SPT profiles for the Area A.

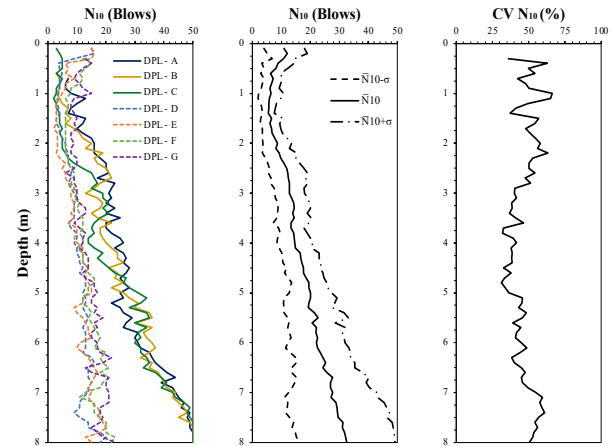


Figure 10. DPL test mean profiles for Area A and Area B combined.

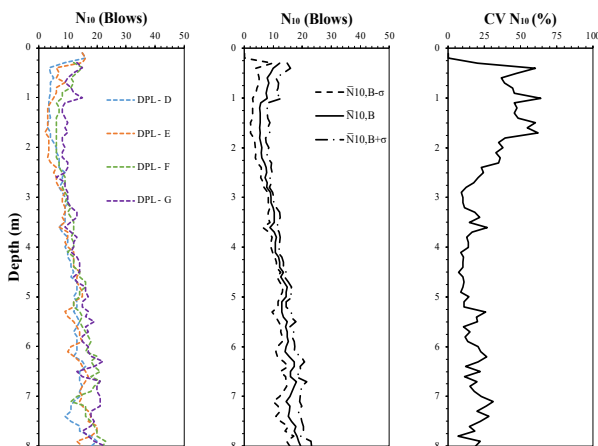


Figure 8. DPL test profiles for the Area B.

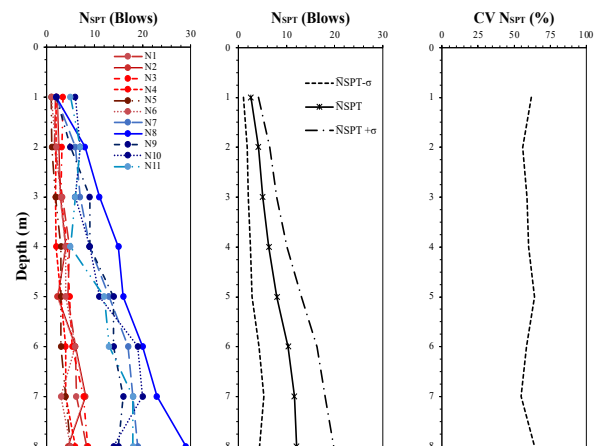


Figure 11. SPT mean profiles for Area A and Area B combined.

4.2 DPL and SPT Correlations

It was possible to correlate N_{SPT} and N_{10} indexes based on the average values. Figure 12 shows the profiles of the average values of the SPT and DPL tests. It is possible to see the similar behavior, with a gradual increase in resistance along the depth.

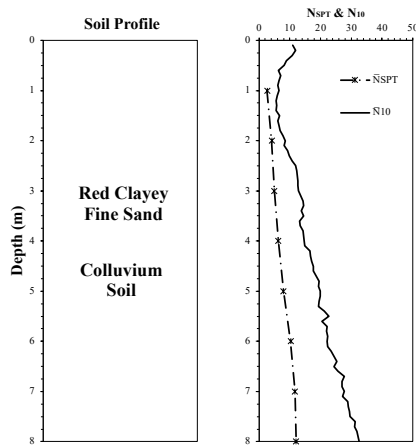


Figure 12. SPT versus DPL mean profiles.

It was possible to propose a correlation between the \bar{N}_{SPT} and \bar{N}_{10} indexes using the linear relationship proposed by Equation 1. The correlation methods (i) to (iv) were adopted, as proposed by Sanchez et al. (2011) and Aguiar et al. (2018).

As can be seen in Table 2, the proposed correlation methods showed a high coefficient of determination (R^2) for all the correlations. According to these values, it was possible to observe that the correlation method (ii) showed the best correlation between the indexes. Figure 13 shows the linear correlation using correlation method (ii).

Table 2. Summary of correlations between N_{SPT} and N_{10} indexes.

Correlation Method	R^2	Equation
(i)	0.9417	$N_{10} = 2.2844 N_{SPT} - 0.2301$
(ii)	0.9617	$N_{10} = 2.5164 N_{SPT} - 1.0274$
(iii)	0.9617	$N_{10} = 7.5493 N_{SPT} - 3.0822$
(iv)	0.8816	$N_{10} = 2.0776 N_{SPT} + 0.7708$

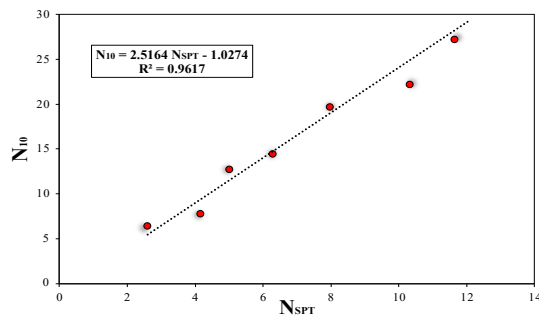


Figure 13. Linear correlation using the correlation method (ii) between N_{SPT} and N_{10} indexes for all the data from Area A and Area B combined.

4.3 Pile Bearing Capacity Prediction

4.3.1 Ultimate Bearing Capacity

To incorporate the 8 m piles into the analysis, \bar{N}_{10} values were adjusted by depth to estimate values for the 9 and 10 m depth by the correlation method (ii), as they are necessary for the prediction methods. The coefficient of determination (R^2) higher than 0.97 justified it.

The ultimate bearing capacity (Q_U) values for the 6 and the 8 m long Mega piles tested in natural conditions determined by the Aoki & Velloso (1975) and Décourt & Quaresma (1978) methods are shown in Tables 3 and Table 4. The \bar{N}_{SPT} values used in both methods were obtained by the equation presented in Figure 13, Table 2 and \bar{N}_{10} values.

The parameters $F_l = 2.5$ for the Aoki & Velloso method (1975), to represent a concrete driven pile proposed by Monteiro (1997) and Pagnussatti & Santos (2011) and a soil with a mix of sand and clay proposed by the method were adopted.

A C coefficient of 250 was adopted for the Décourt & Quaresma method (1978) considering that the behavior of the soil from this site can be understood as a sandy silt by DMT and CPT data (Rocha 2018). Both coefficients α and β were assumed equal to 1.0 as proposed by Joppert Junior (2007) and Mantovani et al. (2020) for driven piles.

Table 3. Ultimate bearing capacity prediction for 6 m long Mega piles for the study site.

Prediction Method	Ultimate bearing capacity (kN)				
	SPT	DPL _i	DPL _{ii}	DPL _{iii}	DPL _{iv}
Aoki & Velloso	169.8	164.9	165.1	171.5	164.7
Décourt & Quaresma	174.8	174.8	174.6	180.3	175.0

Table 4. Ultimate bearing capacity prediction for 8 m long Mega piles for the study site.

Prediction Method	Ultimate bearing capacity (kN)				
	SPT	DPL _i	DPL _{ii}	DPL _{iii}	DPL _{iv}
Aoki & Velloso	242.5	252.5	235.2	258.7	247.5
Décourt & Quaresma	251.5	265.2	266.4	270.3	258.6

Figure 14 shows the variation of the ultimate bearing capacity along the depth for the SPT and DPL assuming the correlation method (ii) to estimate DPL values based on SPT data, since it was considered the most representative of the correlation methods.

The analysis of the data for the 6 m long piles, indicated that the mean values are between the piles E18 and E21, with average values 26% higher and 24% lower respectively, considering the two prediction methods. The average values are between the piles E1, E3, E23 and E26 for the 8 m long piles, ranging from 9 to 39% based on the Aoki & Velloso (1975) method and 22 to 56% based on the Décourt & Quaresma method (1978).

The Aoki & Velloso method (1975) showed better predictions for both cases (6 and 8 m long piles) when comparing to the reference values. On average, the predictions by the Aoki & Velloso method (1975) are in line with that verified by Saraiva Junior & Garcia (2023) for the ultimate bearing capacity prediction using the DPL.

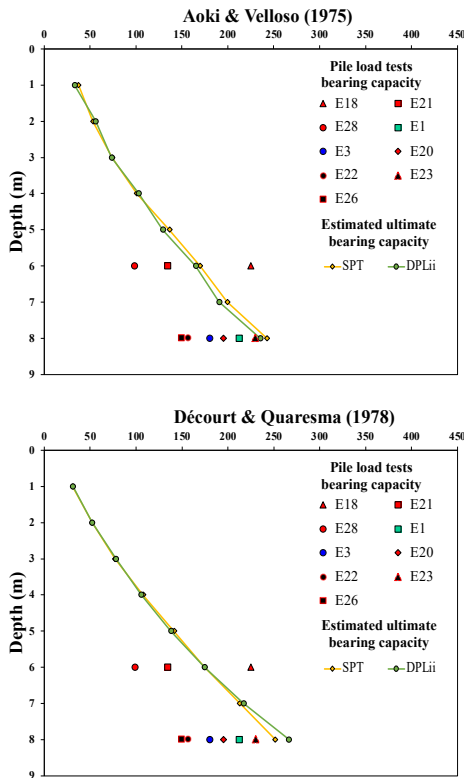


Figure 14. Ultimate bearing capacity (Q_u) estimated assuming correlation method (ii).

4.3.2 Ultimate Bearing Capacity Possible Interval

An analysis of the ultimate bearing capacity was carried out considering the range of variation of the mean values (\bar{N}_{10}) around the standard deviation (σ), as it was shown that the study site has great site variability.

It was possible to incorporate the site variability into the ultimate bearing capacity prediction, by verifying a possible range of occurrence of the ultimate bearing capacity. To do it, the SPT and the DPL data were estimated, assuming the correlation method (ii) for DPL data.

It was possible to verify from this analysis based on the DPL data that the ultimate bearing capacity of tested piles are within the variation ranges considering the site variability. The lower limit values ($\bar{N}_{10} - \sigma$) vary from 15 to 20% in relation to the lower values for both depths. The upper limit values ($\bar{N}_{10} + \sigma$) vary by 5% in relation to the maximum value of 224.7 kN for the 6 m long piles, while for the 8 m long piles the curve is wider than the reference values. It can be related either to the estimative of the \bar{N}_{10} values to incorporate all the piles (both the 6 and the 8 m long) into the analyses or due to the great site variability.

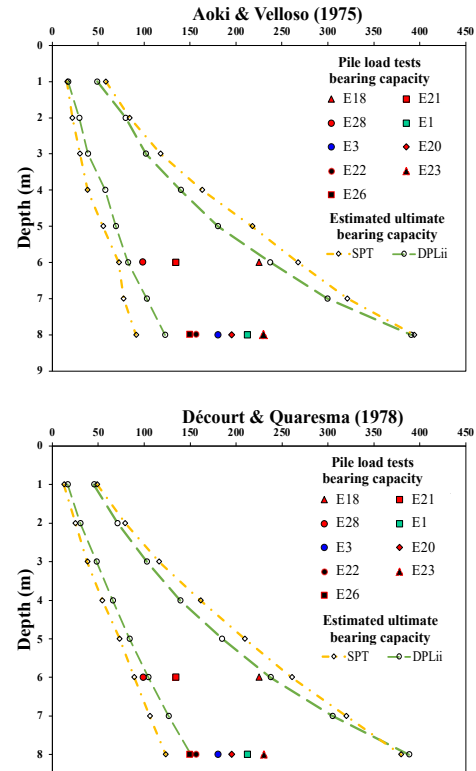


Figure 15. Ultimate bearing capacity (Q_u) interval estimated assuming correlation method (ii).

Figures 16 and 17 show the difference between the values obtained by the pile load tests and those estimated by the semiempirical methods (Aoki & Velloso 1975 and Décourt & Quaresma 1978). Limits of 2.0 and 0.5 (in black) proposed by Schnaid & Langone (2013) for the predicted load tests were used for the comparison.

For the 6 m long Mega piles, when considering the site variability using the standard deviation (σ), most of the predicted results are within the limits suggested by these authors as a classically adopted limit in Brazilian engineering practice.

For the 8 m long piles, the upper limits when considering the site variability are not within the limits proposed by Schnaid & Langone (2013). A limit of 2.6 (in red) was considered more appropriate for the considered piles. It is indicative that the site variability is greater than those observed in Brazilian engineering practice.

Figures 16 and 17 also present, in the green lines, the limits presented in Figure 4 for both 6 m and 8 m long piles. The variability of the in situ tests are within the variability of the pile load tests for the 6 m long piles, as the limits in black and green are similar. However, the variability of the in situ tests is greater than the variability observed in the pile load tests for the 8 m long piles, as the upper limit in black is more wide-open, something that can be related to the estimative in the \bar{N}_{10} values or to the great site variability, characterizing the importance of considering the site variability in foundation engineering design.

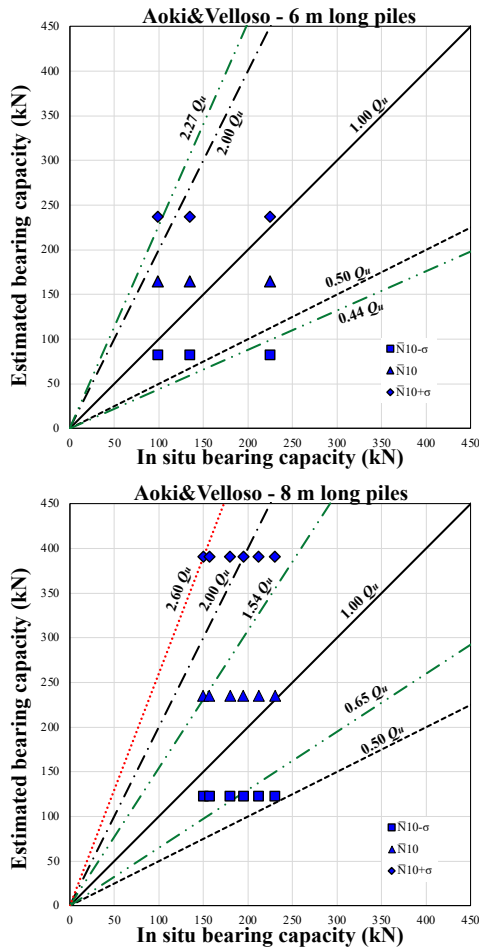


Figure 16. In situ ultimate bearing capacity and estimated ultimate bearing capacity based on DPL data by the Aoki & Velloso method (1975).

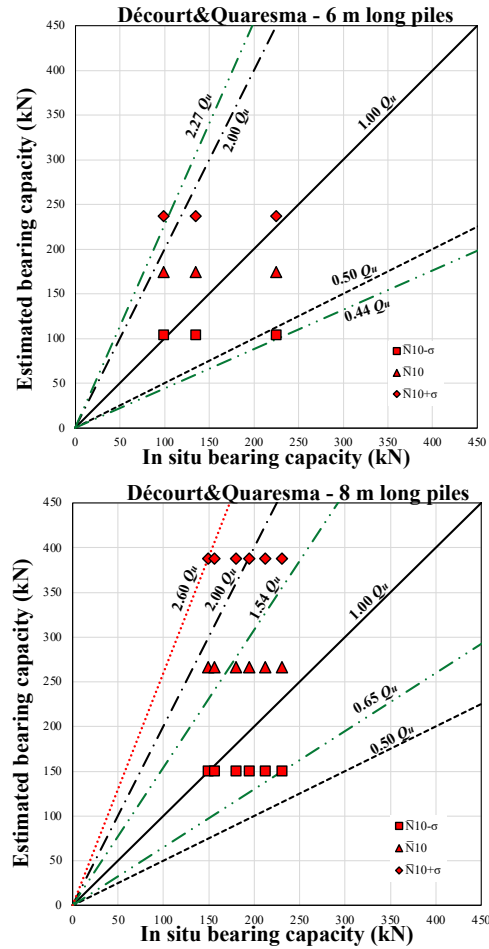


Figure 17. In situ bearing capacity and estimated bearing capacity based on DPL data by the Décourt & Quaresma method (1978).

5 CONCLUSIONS

The DPL is an interesting alternative in situ test for both site characterization and to use in foundation engineering design, mainly due to the test being easily accessible even in sites with difficult access. The aim of this study was to evaluate its applicability in foundation reinforcement projects by Mega piles in a site with high variability.

Data analysis showed that there is a good linear correlation between the N_{10} and N_{SP7} indexes for tests carried out at the studied site. It was possible to verify that the use semiempirical methods based on DPL data mean values underestimate the load bearing capacity by around 20% and overestimate it by 30% in relation to the reference pile load tests. It is worth emphasizing the need to carry out more pile load tests and in situ test campaigns to better define the parameters and indexes of semiempirical methods for Mega piles.

It should also be mentioned that although DPL has several advantages, SPT should not be ignored, since it is important to collect samples and validate the correlations adopted.

Finally, this study sought to verify the importance of adequate site characterization in the foundation design. It was possible to verify that taking geotechnical variability into account can greatly influence decision-making and the results obtained. In addition, the unsaturated condition and seasonality variability, although not presented and discussed, must also be considered in foundation design in unsaturated tropical soils. Therefore, the planning of site characterization has to take into account aspects of site variability and how to incorporate them into the design, since it is important in foundation design and geotechnical works.

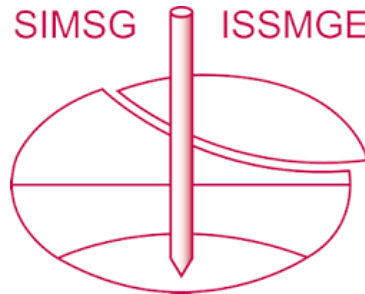
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