Evaluation of Micropiles Load Capacity and Execution Parameters Correlations

Fernando Feitosa Monteiro (1); Yago Machado Pereira de Matos (2); Alfran Sampaio Moura (3); Renato Pinto da Cunha (4); Marcos Fábio Porto de Aguiar (5)

(1) Ph.D Candidate, University of Brasília, Brazil, engffmonteiro@gmail.com
(2) Ph.D Candidate, University of Brasília, Brazil, yago_mpm@hotmail.com
(3) Ph.D in Geotechnical Engineering, Federal University of Ceará, Brazil, alfransampaio@gmail.com
(4) Ph.D in Geotechnical Engineering, University of Brasília, Brazil, rpcunha@unb.br
(5) Ph.D in Geotechnical Engineering, Federal Institute of of Ceará, Brazil, marcosfpa@hotmail.com

Abstract

Foundation design project starts with adequate planning of geotechnical investigation, posteriorly, the most suitable foundation solution is chosen for the site. Foundation performance is verified from the executive control. The executive performance of micropiles is usually determined by static load tests. This paper presents a study on the correlation of micropiles load capacity and monitored variables that are measured during the execution of eight micropiles, using a digital speedometer. Static load tests were conducted on micropiles with 410 and 350 mm diameter. On the static load tests that did not present a visible failure load, Van der Veen`s method (1953) was applied to estimate the failure load. The monitored variables and load capacity presented a good concordance.

Keywords: Load Capacity, Field Parameters, Static Load Tests

Resumen

El proyecto de diseño de la fundación comienza con una planificación adecuada de la investigación geotécnica y, posteriormente, se elige la solución de base más adecuada para el sitio. El desempeño de la fundación se verifica desde el control ejecutivo. El rendimiento ejecutivo de los micropilotes suele estar determinado por las pruebas de carga estática. Este artículo presenta un estudio sobre la correlación de la capacidad de carga de micropilotes y las variables monitoreadas que se miden durante la ejecución de ocho micropilotes, utilizando un velocímetro digital. Las pruebas de carga estática se realizaron en micropilotes con 410 y 350 mm de diámetro. En las pruebas de carga estática que no presentaron una carga de falla visible, se aplicó el método de Van der Veen (1953) para estimar la carga de falla. Las variables monitorizadas y la capacidad de carga presentaron una buena concordancia.

Palabras clave: capacidad de carga, parámetros de campo, pruebas de carga estática

1 Introduction

The comprehension of foundation behavior is one of the great challenges facing foundation engineering, particularly with regard to bearing capacity. The technical community uses analytical tools as common practice to determine the forces that act on deep foundations [1]. Among the many features of the foundation engineering, the executive control of piles is one of the subjects that demands a special regards. The design and the type of foundation depends on the soil profile where the foundation is executed and on the acting load, in addition to other variables. On the design phase, the foundation behavior is usually evaluated by an estimative of load capacity from empirical, semi-empirical and theoretical methods [2]. The performance control is a crucial procedure for site verification of piles performance and such procedure depends on the type of pile. In the design phase, the executive control occurs by means of semi-empirical formulations used in the prediction of pile load capacity. Those formulations use values of static penetration resistance or dynamic penetration resistance obtained in cone penetration tests (CPT) and standard penetration tests (SPT) respectively, where the SPT is the most utilized in Brazil [2]. The root pile is a bored micropile with grouting, where an injection of compressed air is applied on the top of the pile to mold the mortar pile shaft. The executive process of root piles is composed of the following phases: pile pointing, drilling, reinforcement bar placement, mortar filling, drill pipe removal and application of compressed air (Figure 1).

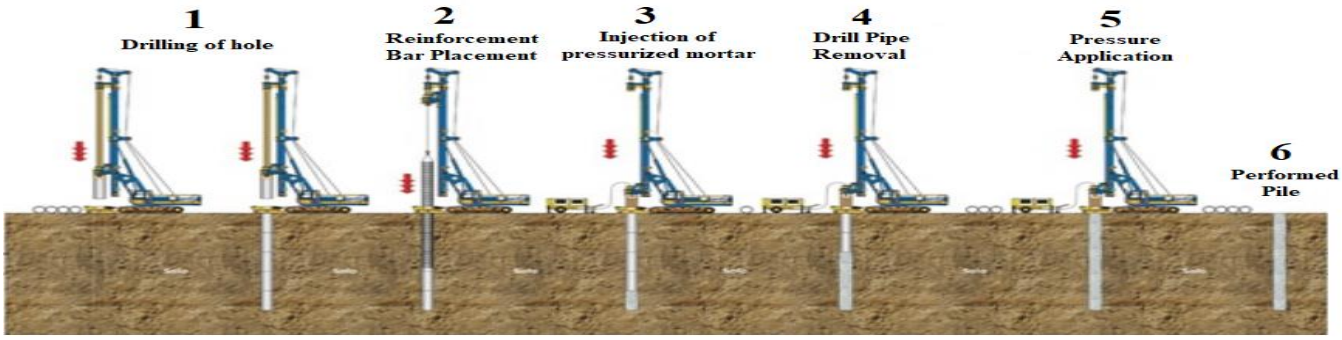

Fig. 1 –Root pile execution stages

2 Case Study

The case study takes place in five different construction sites in the city of Fortaleza, in Brazil, as shown in Figure 2. In those sites, root piles were performed as the foundation of residential buildings. Table 1 presents geometric and executive data of the performed piles in the research sites. The piles studied on the research presented lengths varying from 7,7 to 26 m and diameters of 0,350 and 0,410 m. Injection pressures of 300 and 400 kPa were observed during pile execution. Piles 1 and 2 were executed in Site 1, piles 3 and 4 were performed in Site 2, pile 5 was executed in Site 3 and pile 6 performed in Site 4; Piles 7 and 8 were executed in Site 5. All the analyzed piles in this paper were executed by the same company, same machine type and operator, aiming to minimize the errors of the proposed methodology. The constructive methodology of all the piles followed the execution stages described in Figure 1, where the machine advances with the rotation of the drill bit using water as a drilling fluid.

Table 1 – Geometric and executive data of studied piles

Data	Sites							
	1		2		3	4	5	
Pile	1	2	3	4	5	6	7	8
L (m)	7,7	7,7	15	15	26	12	16	12
D (m)	0,410	0,410	0,410	0,410	0,410	0,350	0,410	0,410
Injection Pressure (kPa)	400	400	300	300	300	300	300	300
Work Load (kN)	1000	1000	1200	1200	1400	800	1200	1200

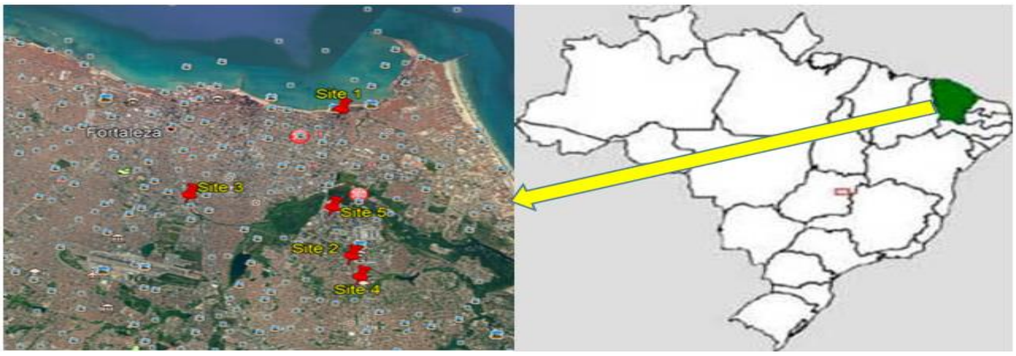

Fig. 2 – Site Locations

The data collection of geological and geotechnical information of the research sites (Sites 1 to 5) was based on data collected through standard penetration tests reports, whose procedures followed the Brazilian standard [3]. In total, 30 standard penetration tests were performed; 5 SPT tests were performed on Site 1 and 2, on Sites 3 and 5, 8 SPT tests were executed, and on Site 4 a total of 4 SPT tests were performed. Figure 3 presents the average SPT results for each site. It is worth mentioning that the SPT tests were performed manually and energy correction factors were not applied. The penetration resistance index value (N_{SPT}) was limited to 60, in depths that present high N_{SPT} values, where the pile execution continued without additional difficulties.

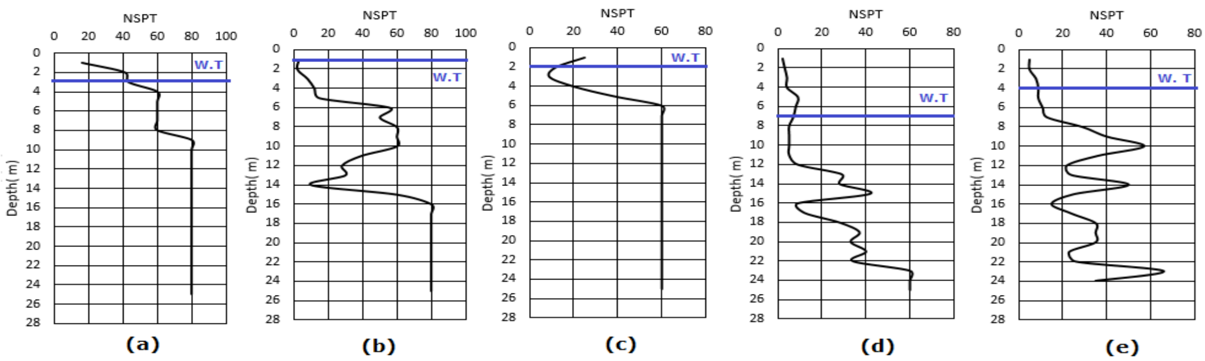

Fig. 3 –Average SPT results for each site – (a) Site 1; (b) Site 2; (c) Site 3; (d) Site 4; (e) Site 5

Site 1 has a clayey silt soil layer with 5m thick, followed by siltstone and sandstone layers, the water table was observed at 3 m depth. Site 2 presents a 12 m thick clayey sand layer, followed by a silty clay layer with thickness varying from 3 m to 5 m, the water table is located at 1 m depth. Site 3 presents a silty sand layer with extension of 2 m, followed by a silt clay layer with 9 m thickness, on the profile base, a magmatic gneiss rock is identified. The water table is verified at 2 m depth. Site 4 features a layered subsoil with silty sand and clayey sands along the entire depth, the water table is observed at 7 m depth. Site 5 presents a silty sand layer with 2 m thick, followed by a clayey silt layer with 9 m thickness and a 12 m of sandy clay, the water table is located at 4 m depth. Site 1, 2 and 3 presented soils with high N_{SPT} values, on ther hand, sites 4 and 5 presented variable NSPT values.

In all sites, monitored piles were submitted to axial static load tests, according to the Brazilian standard [4]. Pile load tests were carried out in 10 load stages, in which each stage correspond to 20% of the pile work load. A hydraulic jack of 3000 kN was used for loading during the execution of the load tests. Load measurement was performed with a 3000 kN load cell with a sensitivity of 1 kN. Four extensometers were placed on the base plate of the hydraulic jack to measure the pile settlement. The reaction system consisted of 4 piles of the same type as the test pile, presenting the same diameter and length. A total of 8 static load tests were carried out in those sites during the research. Static load test results are presented in Figure 4a, 4b, 4c, 4e, 4f and 4g.

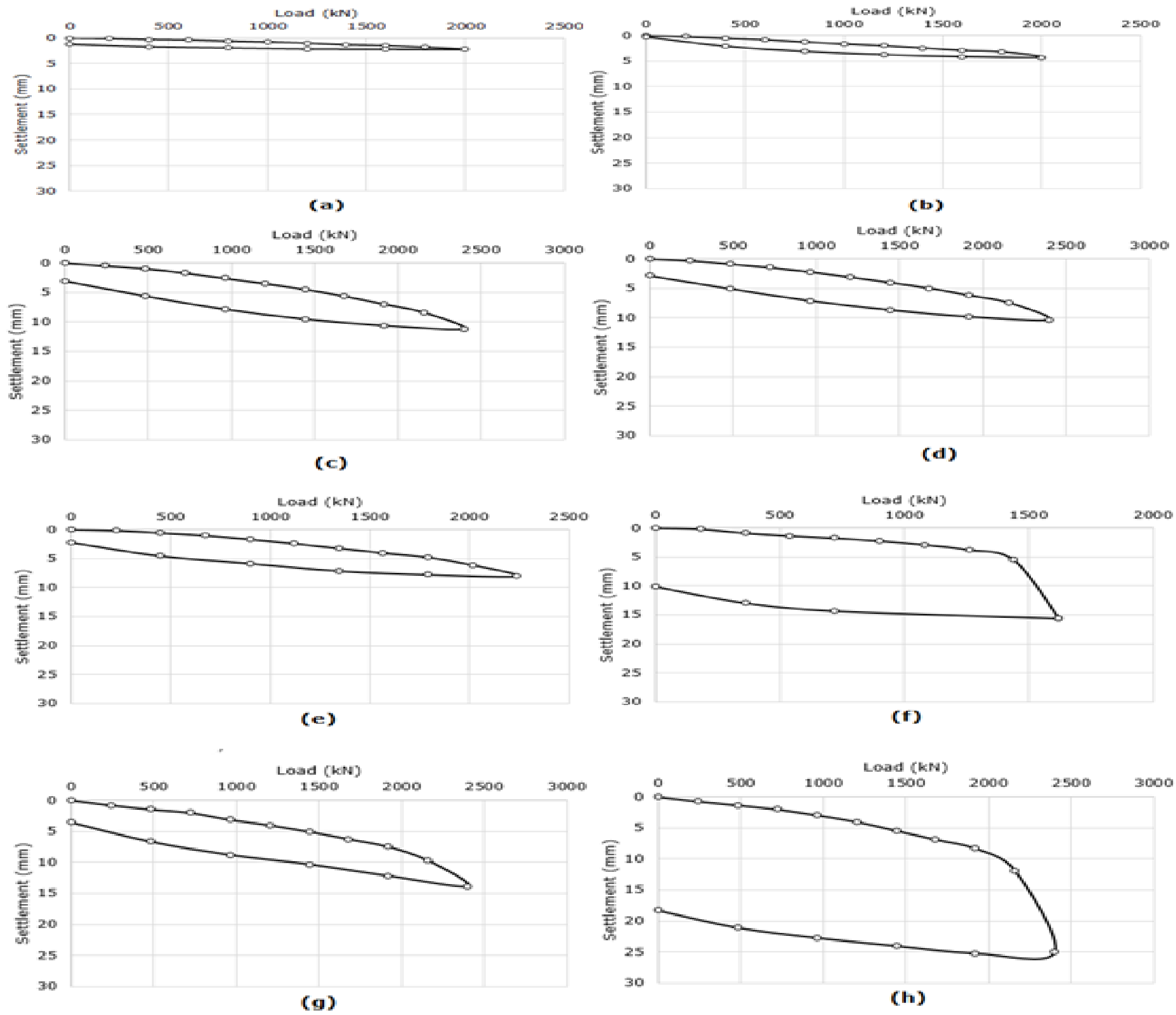

Fig. 4 – Load Tests Results – (a) Pile 1; (b) Pile 2; (c) Pile 3; (d); Pile 4; (e) Pile 5; (f) Pile 6; (g) Pile 7; (h) Pile 8

It is worth mentioning that in static load tests that did not present physical failure, the established Van der Veen method (1953) [5] was used for failure load extrapolation, which is the case of piles 1, 2, 3, 4 and 5. For piles 6, 7 and 8, in which it was possible to visualize the failure load value, when large displacements were observed for small load increments, no extrapolation methods were required. Thus, the adopted failure load values of the studied piles are described in Table 2. The determination of failure load without the soil-pile system physical failure is a controversial issue in foundation engineering, although Van der Veen's (1953) methodology has great national acceptance [6].

Table 2 – Geometric and executive data of studied piles

Site	Pile	Q_{ult} (kN)
1	1	3000
	2	3200
2	3	3100
	4	2900
3	5	2800
4	6	1550
5	7	2450
	8	2150

3 Monitoring Process

Moura et al. (2015) introduced the monitoring process used in this paper, which consists on the monitoring of variables related to pile geometry and to the soil profile that the pile was executed, such as: drill's linear velocity (v_b), advance velocity (v_a), pile length (L), tip area (A_p), lateral resistance index ($N_{SPT,lat}$) and tip resistance index ($N_{SPT,\ tip}$). The digital speedometer, used in the research (Figure 4), provides measurement of the linear speed of the drill rotator, the linear distance traveled by the drill rotator, and the stopwatch function.

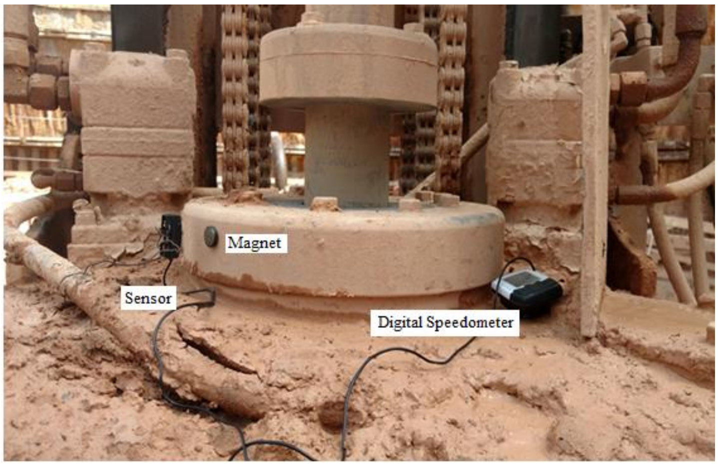

Fig. 4 – Monitoring equipment

The drill rotator diameter is digitally inserted in the speedometer, in such a way that the sensor records each complete lap performed by the drill rotator (magnet). This procedure allows to record the linear distance traveled by the drill rotator for a determinate period of time.

The drilling rod used to drill the last meter length of the pile was then, marked with sections of 10, 20 and 20 cm.The elapsed time to drill the length between 2 marked sections was recorded in order to determine the advance velocity (v_a). The other measured variable is the drill rotator linear velocity (v_b), which is measured for the same section.

The linear distance traveled by the drill rotator between 2 marked sections is divided by the circumference length of the drill rotator, thus, obtaining the number of laps performed by the drill rotator during the drilling of the mentioned section. From the elapsed time during the drilling of this section and the number of laps performed by the drill rotator, it is possible to determine the number of laps per minute, that is, the frequency (f) of the drill rotator. Measurements of some relevant variables during pile execution, which would later be submitted to static load test were recorded. Variables that were related to pile performance were monitored.

The results of this monitoring phase are featured in Table 3, where the evaluated variables are: excavation length and excavation time, from which the advance velocity (v_a) was obtained; drill rotator frequency, which is determined from the number of rotations performed by the rotator during the excavation time; drill bit linear velocity (v_b), which is associated with the drill rotator frequency; ($N_{SPT,\ tip}$) and the average penetration resistance index along the pile ($\bar{N}_{SPT,\ lat}$).

It can be observed that during the execution of the pile last meter, the measured excavation length during monitoring varied between 0,35 and 0,7, those sections were segmented for a better understanding of the monitored variables behavior. At first, it was sought to carry out the monitoring in sections of: 10 cm, 20 cm and 20 cm. However, due to unforeseen events that occurred in the field, it was necessary to make changes in the length of the monitored sections.

For piles 1 and 2 executed in similar stratigraphic profiles (Site 1), higher excavation times are observed when compared to the other piles, whose motive is attributed to the pile tip seated on rock profile. Thus, for these piles, lower advance velocities are verified. However, the drill rotator frequency, which is directly associated with the drill bit linear velocity, presents similar values (which decrease 23.0%) when compared with values for piles 4 and 5. Pile 5 (Site 3), which has the tip supported in magmatic gneiss, and which has an average penetration resistance index along the shaft similar to piles 1 and 2, which also have the tip supported on rocks. In Site 4, where pile 6 was performed, a reasonable compliance with pile 8 is verified when advance velocity is evaluated. Piles 6 and 8 are embedded in soils stratigraphy that alternate between silty sandy, sand, clay and sandy clay. These piles present a 7.4% variation when compared to the respective advance velocities, which is smaller for pile 8. This is due to the fact that the penetration resistance index at the tip of the pile 8 is higher than pile 6. Thus, it is observed a correlation between these two variables, in such a way that, the higher the penetration resistance index, the lower the advance velocity, presenting an inversely proportional ratio.

Table 3 – Monitoring results

Pile	Excavation length (m)	Time (s)	Advance Velocity (m/s)	Frequency (Hz)	Linear Velocity (m/s)	$N_{SPT, tip}$	$N_{SPT, lat}$
1	0,10	38,00	2,63E-03	2,01	1,95	60	50
	0,20	51,00	3,92E-03	2,50	2,44		
	0,20	78,00	2,56E-03	2,15	2,09		
	0,20	72,00	2,78E-03	2,25	2,19		
2	0,10	27,00	3,70E-03	1,76	1,72	60	52
	0,20	50,00	4,00E-03	2,67	2,60		
	0,20	56,00	3,57E-03	1,36	1,33		
	0,20	54,00	3,70E-03	2,65	2,58		
3	0,10	11,22	8,91E-03	2,55	2,48	60	33
	0,10	8,27	1,21E-02	1,15	1,12		
	0,20	19,28	1,04E-02	0,99	0,96		
4	0,10	4,76	2,10E-02	2,00	1,95	60	32
	0,20	9,78	2,04E-02	1,95	1,90		
	0,20	15,84	1,26E-02	1,20	1,17		
5	0,10	18,59	5,38E-03	1,54	1,50	60	52
	0,20	34,30	5,83E-03	1,94	1,89		
	0,20	43,19	4,63E-03	1,76	1,72		
6	0,15	29,00	5,20E-03	3,99	3,89	10	6
	0,20	43,00	4,70E-03	4,06	3,95		
7	0,30	30,00	1,00E-02	2,05	1,99	39	22
	0,20	27,00	7,40E-03	2,44	2,37		
8	0,30	38,00	7,90E-03	1,61	1,57	22	22
	0,20	44,00	4,50E-03	4,38	4,27		

Piles 7 and 8 were executed in Site 5, whose soil profiles are similar. However, it is worth mentioning that the tip of pile 7 is supported on a silty clay soil and the tip of the pile 8 is seated in a clayey sand soil. In these cases, the tendency indicated by [1] regarding the advance velocity and the tip penetration index of the pile is not observed, because the soil where pile 7 is supported presents a tip penetration index higher than the soil below pile 8, as well as the advance velocity. By comparing piles 6, 7 and 8, high frequency values were observed, which were already expected due to the direct frequency relationship with the drill bit linear velocity (v_b). Thus, it is possible to observe, that the higher the frequency, the greater the drill bit linear velocity and the smaller the soil N_{SPT}.

Based on the above, it is possible to notice that pile shaft load capacity is inversely proportional to the drill bit linear velocity (v_b) and the frequency. Thus, for example, the greater the pile shaft area

(U x L) and the average penetration resistance index along the pile, the greater the pile skin friction resistance. On the other hand, the greater the drill bit linear velocity, the lower the shaft resistance, as seen in Figures 5a, 5b and 5c.

Likewise, for the tip load capacity, it is observed that the advance velocity is inversely proportional to the pile tip load capacity. Thus, the higher the advance velocity (v_a), the lower the tip resistance, and the higher the tip area (Ap) and the tip resistance index ($N_{SPT,\ tip}$), the higher the tip resistance as seen in Figures 6a, 6b and 6c. It is worth mentioning that the variables tip area (Ap) and pile shaft area (UL) did not present significant influence in the estimation of the load capacity of the monitored piles.

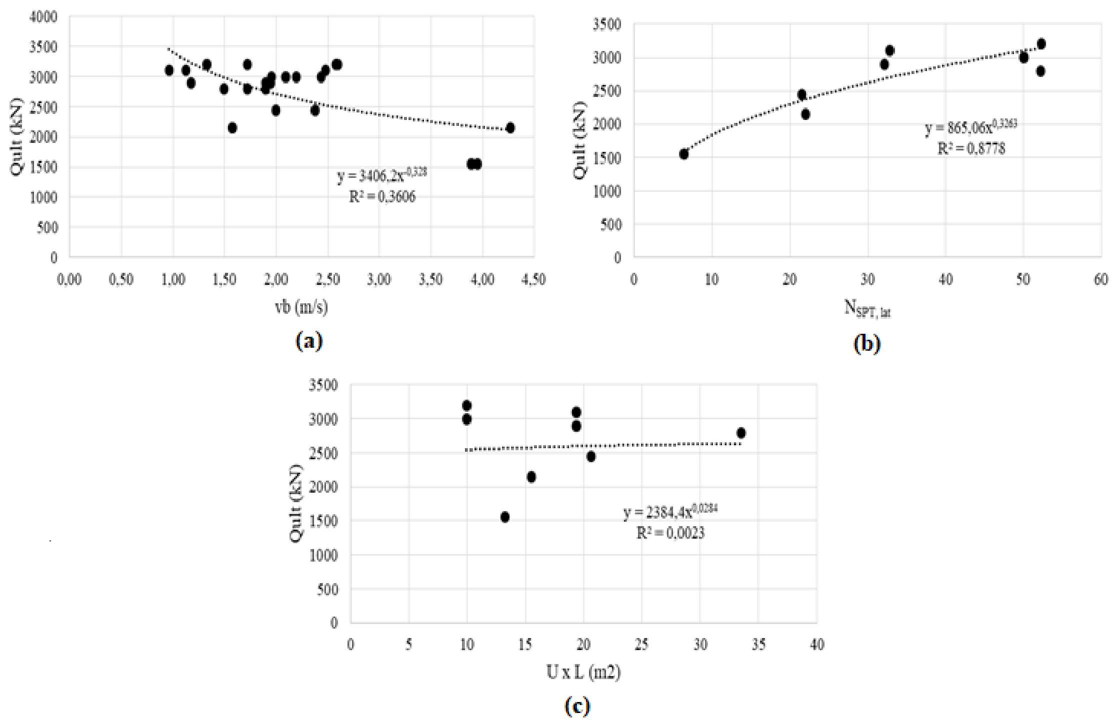

Fig. 5 – (a) Q_{ult} x v_b ; (b) Q_{ult} x $N_{SPT,\ lat}$; (c) Q_{ult} x U L

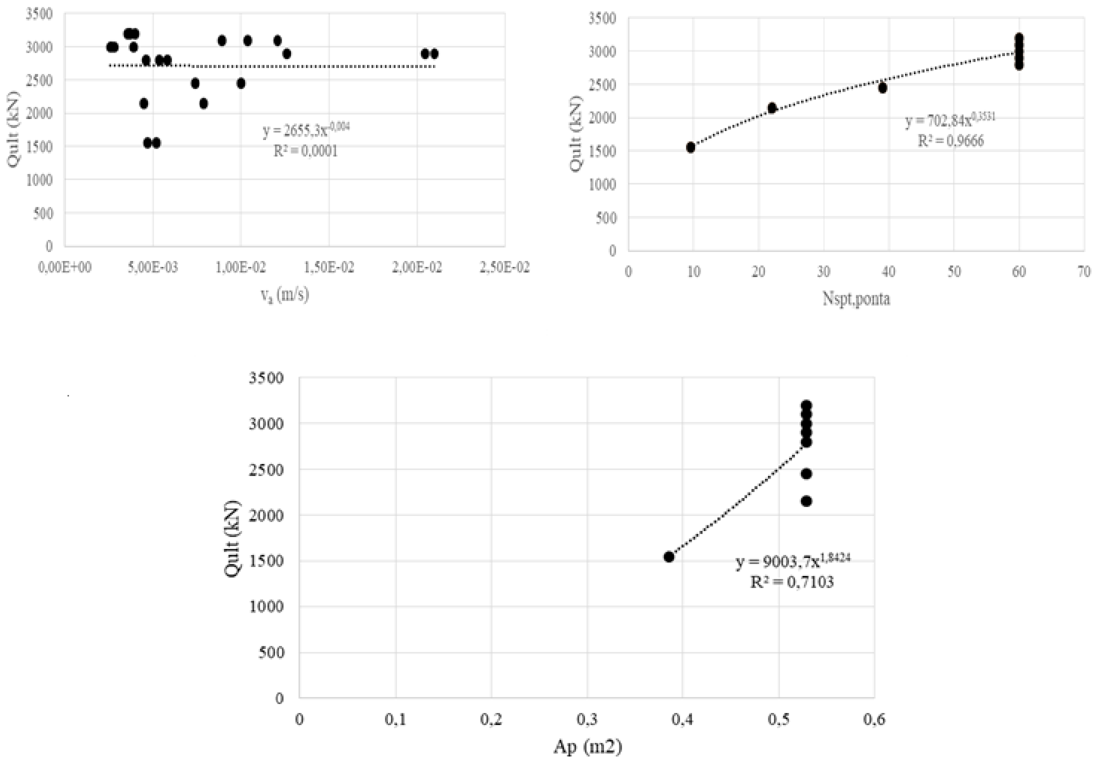

Fig. 6 – (a) Q_{ult} x v_a ; (b) Q_{ult} x $N_{SPT, tip}$; (c) Q_{ult} x Ap

4 Conclusions

This paper presented an alternative monitoring method for micropiles performance control, in order to evaluate pile load capacity during the pile execution. The monitored variables and load capacity presented a good concordance. From the study, it was possible to observe that:

(1) The higher the average penetration resistance index along the pile shaft ($N_{SPT, lat}$), the lower the linear velocity of the drill (v_b), thus, it is observed that the relationship between these variables is inversely proportional.

(2) It should be noted that the advance velocity variable (v_a) did not present a significant influence on the monitored piles load capacity.

(3) A directly proportional relationship between the frequency and the drill linear velocity (v_b) is observed. It was also possible to evaluate that the skin friction resistance is inversely proportional to the linear velocity of the drill (v_b) and the frequency.

(4) The variables tip area (Ap) and pile shaft area (UL) did not present significant influence in the the load capacity of the monitored piles

(5) The advance velocity is inversely proportional to the pile tip resistance.

(6) The higher the tip area (Ap) and the tip resistance index ($N_{SPT,\ tip}$), the higher the tip resistance.

5 References

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