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CHANGE OF EXECUTIVE METHODOLOGY IN FOUNDATION DESIGN DURING THE EXECUTION PHASE OF A CONSTRUCTION PROJECT IN CANAÃ DOS CARAJÁS-PA

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Abstract: The choice of foundation type is one of the main stages in the preparation of a foundation project. A quality geotechnical investigation, a good study of the boundary conditions and the execution of a load test campaign before issuing the executive design of the foundation are essential for providing the best possible information and for the project to be properly dimensioned. The aim of this article is to present a case study of a construction project in Canaã dos Carajás-PA, where part of the executive foundation project consisted of excavated piles with the use of stabilizing fluid (Ø1.0m), but during the staking process, the construction company confirmed that it is possible to replace the execution method with the monitored continuous auger type (Ø1.0m), with the piles mobilizing the same or greater geotechnical load capacity, reducing the useful length of the shaft, the weight of the steel reinforcement and the volume of concrete, as well as, above all, the execution time. After confirming that it was possible to achieve the useful lengths dimensioned by semi-empirical methods, dynamic load tests and static load tests were carried out, which proved that the minimum load capacity required had been mobilized. The replacement of the foundation execution methodology led to a reduction of 2.5% in the length drilled, 26.9% in the weight of steel, 4.3% in the volume of concrete and 62.5% in the execution time. It can therefore be seen that advances in the technology of drilling rigs that execute monitored continuous helical piles have increased drilling capacity in competent soils, promoting the viability of the methodology in scenarios that were utopian years ago.

Keywords: pile - monitored continuous auger - mining - static load tests - dynamic load test

INTRODUCTION

Foundation engineering is growing exponentially with the advance of technology. Young engineering students have been trained with a less aggressive view of the foundations stage of the project, as the dissemination of technical knowledge and awareness of the importance of this stage of the construction has been intense on social networks and universities, a beneficial factor for the future of engineering in Brazil.

The search for optimizations in every stage of the project is one of the main tasks of an engineer, and in the area of foundations, this gain can be immense. When designing the foundations, based on the complexity of the structure (large loads and purpose), the first step for a geotechnical engineer is to request a geotechnical investigation campaign to obtain the characteristics of the soil at the building site.

In addition to the geotechnical investigation, before determining the type of foundation to be adopted for sizing and subsequent execution, it is necessary to analyze the boundary conditions, access and topography of the construction site, in order to make the foundation project technically feasible (VELLOSO; LOPES, 2010).

At this stage, the involvement of the design team and the execution team is crucial: the former seeks solutions based on geotechnical engineering knowledge, and the latter complements this knowledge with aspects relating to the limitations of the equipment that will be involved, access limitations and other aspects inherent to the construction methods (ALONSO, 2019).

Since most of the methods used in Brazil for the geotechnical design of foundations are semi-empirical, and since the behavior of the soil mass is uncertain, it is essential to know the real behavior of the foundation from the application of the loads provided in the designs. This knowledge is obtained by carrying out load tests or dynamic loading tests on an isolated foundation element.

The aim of this paper is to present a case study in which the methodology used to execute the foundations of a construction site located in the region of Canaã dos Carajás-PA was changed during the piling phase, from excavated piles using stabilizing fluid to monitored continuous helix piles.

TYPES OF PILES

EXCAVATED PILES USING STABILIZING FLUID

The Brazilian technical standard 6122 defines it as, "J.2 These are piles excavated using a stabilizing fluid, which can be bentonite mud for drilling or synthetic, natural or modified natural polymers to support the excavation walls. Concreting is submerged, with the concrete displacing the stabilizing fluid upwards out of the hole." (ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS, 2022, p. 72).

Advances in fluid manufacturing technology have led to the replacement of bentonite mud, a product used in the first large-diameter excavated piles that was harmful to the environment, with a polymeric mud made from biodegradable polymers, bringing sustainability to the methodology used in excavated piles. In terms of executive limitations, today in Brazil there are drilling rigs that have the capacity to advance in rocky material with diameters of up to 2.5m, and depths of 50.0m.

MONITORED CONTINUOUS PROPELLER PILES

The Brazilian technical standard 6122 defines it as, "N.2 It is a cast-in-place concrete pile, made by driving a continuous helical auger of constant diameter into the ground by rotation. Concrete is injected through the central shaft of the auger at the same time as it is withdrawn. The reinforcement is always placed after the pile has been concreted." (ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS, 2022, p. 89).

With advances in equipment manufacturing technology, today's drilling rigs in Brazil have automated hydraulics, ensuring greater control and safety during execution, and can drill through soils with NSPT values >50 blows, with a diameter of 1.2m and a length of 40.0m. However, it is not yet possible to embed in rock, only to support it, which, with the correct technique at the concreting stage, without causing dirt at the tip of the pile, guarantees a considerable mobilization of tip resistance.

SEMI-EMPIRICAL METHODS FOR GEOTECHNICAL DESIGN

AOKI-VELLOSO (1975)

Method developed by Nelson Aoki and Dirceu Velloso, and the geotechnical load capacity of the pile is found from Equation 1 below.

$$R = \frac{K \, . \, N_P}{F_1} \, . \, A_P \, + \, U \, . \, \sum \left(\frac{\alpha \, . \, K \, . \, \overline{N}_{SPT}}{F_2} \right) . \, \Delta_L \qquad (1)$$

Where U is the perimeter of the pile cross-section, α and K are factors that depend on the soil type, NSPT is the number of blows for the respective soil layer, ΔL is the pile segment in the respective soil layer, F1 and F2 are correction factors that depend on the pile type, NP is the NSPT value in the soil layer of the pile tip and AP is the area of the pile tip.

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DÉCOURT-QUARESMA (1978)

Method developed by Luciano Décourt and Arthur Quaresma, and the geotechnical load capacity of the pile is obtained using Equation 2 below.

$$R = \alpha . C . \overline{N}_P . A_P + U . \beta . \Delta_L . \left[10 . \left(\frac{\overline{N}_{SPT}}{3} + 1\right)\right] \qquad (2)$$

Where U is the perimeter of the pile cross-section, β and α are factors that depend on the type of soil and the pile, NSPT is the number of blows for the soil layer, ΔL is the pile segment in the respective soil layer, C is a factor that depends on the type of soil, NP is the value resulting from the average of three values obtained at the level of the pile tip immediately above and below it and AP is the area of the pile tip.

ANTUNES AND CABRAL (1996)

Method developed by William Antunes and David Cabral, applicable only to monitored continuous auger piles, where the pile's geotechnical load capacity is obtained using Equation 5 below.

$$R = \beta_2' . N_P . A_P + U . \sum(\overline{N}_{SPT} . \beta_1') . \Delta_L \qquad (3)$$

Where U is the perimeter of the pile cross-section, β'2' and β'1 are factors that depend on the soil type, NSPT is the number of blows for the soil layer, ΔL is the pile segment in the respective soil layer, NP is the NSPT value in the soil layer of the pile tip and AP is the area of the pile tip.

PILE PERFORMANCE TESTS

STATIC LOAD TEST

According to Alonso (2013, p. 2), "The load test consists of loading the pile (in compression, traction or horizontally) with progressive load increments (P) and measuring the corresponding displacements (d), resulting in a load x displacement graph."

The main result of a static load test is the load x settlement curve, which is obtained by collecting the vertical displacement values (settlements) presented for each load applied in the loading and unloading stages. It is from the analysis of this curve that the information regarding the pile's load capacity is obtained.

The test is carried out in loading and unloading stages, which have criteria defined according to the type of loading chosen, slow, fast, mixed, slow cyclic or fast cyclic, with slow being the most widely used in Brazil, standardized by the then current ABNT NBR 16903 (ABNT, 2020).

DYNAMIC LOADING TEST

According to Alonso (2019, p.141), "the dynamic loading test is somewhat similar to the rapid cyclic static load test. The objective is to obtain a mobilized load x maximum dynamic rebound curve, referring to a series of pylon blows with increasing energies."

The test consists of applying a series of blows with a pylon of known mass to the top of the pile. With each blow of the pylon, there is an axial displacement caused by the pylon falling from a defined height. The load mobilized in each blow is obtained by monitoring the blow or by interpreting the displacement x time curve. The displacement can be measured from the pile driving control by means of a sheet of paper fixed to the pile section or with devices that calculate the displacement x time curve for each blow (ALONSO, 2019).

The test has been carried out in Brazil since the early 1980s, but was only standardized in the country in 2007, with the creation of technical standard ABNT NBR 13208 (2007), which is still in force today.

CASE STUDY

The initial project for a large construction project, located in a mining complex in the city of Canaã dos Carajás-PA, provided for the execution of part of the foundations with piles excavated with stabilizing fluid, dimensioned only from SPT surveys. However, after the start of the project, the low productivity during the execution of these piles led to the presentation of a technical study by the executing company to the end client, about the possibility of changing the execution methodology to monitored continuous auger piles, with the aim of substantially increasing productivity and reducing the inputs inherent in the execution process.

Data from 13 SPT borings, 05 static compressive load tests with slow loading and 03 dynamic loading tests on monitored continuous auger piles and excavated piles using stabilizing fluid from a large construction site were used to carry out this work,

The geotechnical design of the piles was based on semi-empirical methods, using data from 13 SPT borings, chosen because they were carried out within a radius of up to 20.0m from the piling area under study. The global safety factor adopted was optimized according to the technical standard ABNT NBR 6122 (2022).

The validity limits of the NSPT values of each of the semi-empirical methods applied were respected, and the respective safety coefficients were applied to obtain the admissible resistances, which are the resistances presented in this work.

Static load tests and dynamic loading tests were carried out after the methodology was replaced, with loads applied to mobilize 2.0x the working load of the piles, as defined in the site's executive projects. However, load tests 3, 4 and 5 did not reach 2.0x the working load, only 1.6x, due to the possibility of instability in the reaction system.

INITIAL PROJECT

The initial executive project included the construction of 547 excavated piles using stabilizing fluid, distributed over an area of approximately 11,380m², in 06 piled radiers and 04 blocks under piles, with a useful stem length of between 25.0m and 30.0m, fully reinforced, and a provisioned load capacity of 400.0tf.

SOIL CHARACTERISTICS

Thirteen SPT boreholes were used for this study, carried out at different elevations, as shown in Table 2. In general, practically all the profiles indicated the predominant presence of clayey silt (mature residual) in the first 3.0m, followed by silty clay (young residual from 13.0m and saprolite from 18.0m, with shale rock structure) with NSPT varying between 10 and >50 blows, with the subsequent presence of an impenetrable layer, and water level from 9.0m. All the boreholes were stopped due to the impenetrability criteria recommended in the technical standard ABNT NBR 6484 (2020).

GEOTECHNICAL DESIGN USING SEMI-EMPIRICAL METHODS

From the 13 SPT boreholes, geotechnical design was carried out using the methods of Aoki-Velloso (1975), Décourt-Quaresma (1978) and Antunes and Cabral (1996), without the application of safety factors, with characteristic resistance values in tons-force shown in Table 3.

In order to optimize the safety factor used to obtain the admissible load, the method proposed by technical standard ABNT NBR 6122 (2022) was applied, "6.2.1.2.1 Resistance determined by semi-empirical method", governed by Equation 4.

$$P_{adm} = \frac{mín\left[{}^{(R_{se})méd}/_{\xi_1};{}^{(R_{se})mín}/_{\xi_2}\right]}{FS_g} \qquad (4)$$

Prof. (m)	SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10	SP11	SP12	SP13
1	-	-	-	-	-	-	-	-	-	-	3	3	-
2	-	24	-	-	-	-	-	8	-	4	3	4	-
3	-	21	6	9	13	-	9	7	-	4	3	5	7
4	8	16	7	12	10	-	8	6	-	4	3	5	9
5	15	15	7	12	3	9	10	9	-	3	3	4	7
6	18	12	14	13	17	6	9	11	-	5	3	3	10
7	14	8	7	14	22	7	15	10	9	6	3	5	7
8	17	15	13	16	29	10	12	9	9	14	3	9	9
9	19	31	15	14	44	26	13	11	18	15	3	12	13
10	15	27	10	15	34	31	15	11	50	19	8	16	17
11	21	28	15	14	34	32	28	8	50	19	12	19	19
12	19	27	19	14	19	35	31	9	50	25	24	22	23
13	18	50	15	13	22	38	35	11	50	27	26	8	19
14	17	32	17	17	39	21	29	13	50	34	30	18	23
15	22	38	15	11	30	23	32	16	26	39	33	31	25
16	18	45	17	16	27	28	50	23	31	34	34	34	30
17	24	38	28	16	32	30	50	50	33	38	36	35	27
18	20	36	50	14	50	14	49	50	23	38	39	39	29
19	20	50	50	17	50	19	50	13	22	39	36	41	33
20	27	50	50	16	50	22	50	15	19	50	36	44	34
21	24	50	50	15	50	50	50	17	23	50	50	39	43
22	35	IMPORTANT	50	20	50	50	IMPORTANT	18	18	50	50	50	45
23	32		50	15	IMPORTANT	50		18	28	50	50	50	43
24	26		IMPORTANT	13		50		18	25	IMPORTANT	IMPORTANT	50	47
25	31			18		IMPORTANT		40	31			50	50
26	45			19				50	50			IMPORTANT	IMPORTANT
27	50			15				50	50				
28	50			19				50	50				
29	50			18				50	50				
30	IMP.			22				IMP.	50				
31				35					IMP.				
32				30									
33				43									
34				44									
35				50									

Table 2. NSPT values per borehole from depth. Prepared by the author (2024).

Survey	Pile type	Diameter (m)	Useful length (m)	Décourt-Quaresma (1978) (tf)	Aoki-Velloso (1975) (tf)	Antunes and Cabral (1996) (tf)
SP1	HCM	1,0	25,0	934,10	896,95	943,89
SP2	HCM	1,0	25,0	1.293,29	1.138,63	1.276,59
SP3	HCM	1,0	25,0	1.177,05	1.061,53	1.171,19
SP4	HCM	1,0	25,0	641,20	592,54	716,60
SP5	HCM	1,0	25,0	1.424,19	1.178,60	1.301,72
SP6	HCM	1,0	25,0	1.241,98	995,77	1.010,02
SP7	HCM	1,0	25,0	1.407,43	1.092,81	1.134,11
SP8	HCM	1,0	25,0	911,06	885,82	931,48
SP9	HCM	1,0	25,0	1.168,67	1.056,07	1.163,80
SP10	HCM	1,0	25,0	1.211,61	1.076,87	1.187,05
SP11	HCM	1,0	25,0	1.228,36	1.003,05	1.027,61
SP12	HCM	1,0	25,0	1.133,07	1.027,65	1.121,71
SP13	HCM	1,0	25,0	1.140,40	1.039,49	1.142,60

Table 3. Geotechnical design results, semi-empirical methods. Prepared by the author (2024).

Permissible resistance	Pile type	Diameter (m)	Useful length (m)	Décourt-Quaresma (1978)	Aoki-Velloso (1975)	Antunes and Cabral (1996)
Padm (tf)	HCM	1,0	25,0	412,6	381,3	461,1

Table 4: Admissible resistance, after optimizing the safety factor. Prepared by the author (2024).

Load test	Pile type	Survey ref.	Diameter (m)	L (m)	Qmax (tf)	ρ maximum (mm)	ρ residual (mm)
PCE01	HCM	SP8	1,0	27,00	640,0	7,71	4,39
PCE02	HCM	SP11	1,0	25,05	800,0	3,84	1,31
PCE03	HCM	SP1	1,0	25,06	800,0	3,35	1,27
PCE04	HCM	SP10	1,0	27,00	640,0	3,51	0,89
PCE05	HCM	SP6	1,0	25,21	640,0	3,91	1,16
ECD01	HCM	SP8	1,0	23,81	830,5*	9,00	2,00
ECD02	HCM	SP3	1,0	25,35	936,8*	8,00	2,00
ECD03	Excavated with fluid	SP11	1,0	27,38	649,1*	8,00	3,00

Table 5: Load test results. Prepared by the author (2024).

*Maximum load mobilized in the test, obtained by the CAPWAP® method.

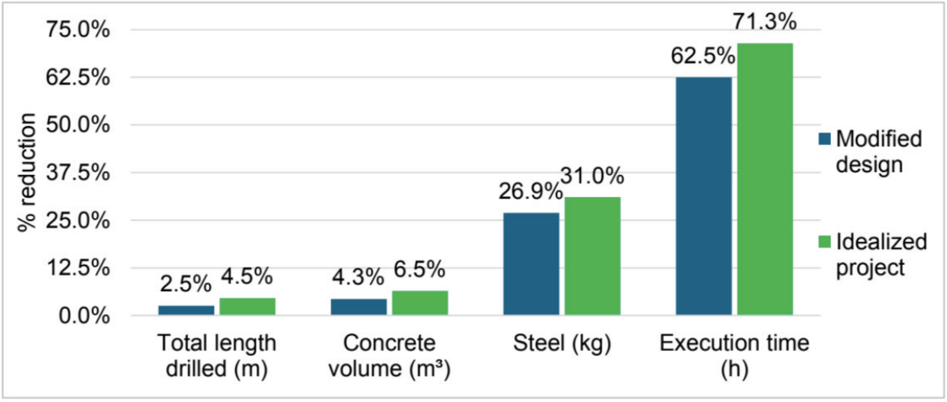

Figure 1. Percentage reductions by indicator between projects. Prepared by the author (2024).

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Where (Rse)méd and (Rse)mín are the resistances determined on the basis of the average and minimum values of the field test results, ξ1 and ξ2 are the resistance reduction factors specified in Table 2 of ABNT NBR 6122 (2022) and FSg is the global safety factor, defined as 1.4.

The results obtained after optimizing the safety factor are shown in Table 4.

RESULTS

Analyzing the strength results of the semi-empirical methods, it can be seen that one of the borehole profiles, SP4, had much lower NSPT values than the others, a fact that substantially influenced the final admissible strength being much lower than the others.

From the results obtained in the geotechnical design, it was found that if the piles were executed as a monitored continuous auger type, they would reach the required load capacity of 400.0tf, with the exception of the Aoki-Velloso (1975) method, which was very close.

Based on this, 6 drilling tests were carried out in the area, with a hydraulic drill with a maximum torque of 280kN.m, in order to check that the required lengths had been reached. In 5 of the 6 holes, the required depths were reached, with the exception of 1 hole, which fell 2.0m short of the target.

Following the success of the test holes, the executive project was revised, modifying the pile methodology but maintaining the quantities and locations. However, 61 piles had already been driven as excavated piles with stabilizing fluid, leaving 487 to be driven as monitored continuous auger piles with a useful length of 25.0m and 12.0m of reinforcement, including the start and shaft.

CARRYING OUT LOAD TESTS

Throughout the execution of the new executive methodology adopted, 05 static compressive load tests were carried out with slow loading and 03 dynamic loading tests, using a 10.5ton hammer with a drop height of up to 2.1m, capturing signals with an electronic device connected to strain transducers and accelerometers. The results obtained are shown in Table 5.

No clear rupture was identified in any of the static load tests carried out and, after analyzing the load x settlement curve, it was found that none of the piles came close to conventional rupture, due to the small displacements and the fact that the curve did not tend to form an asymptotic graph with a vertical line, thus confirming the load capacities provided.

IMPACT OF THE CHANGE IN FOUNDATION METHODOLOGY

After the piles were completed, a survey was carried out and some variables compared between the initial executive project, the executive project after the change in methodology, and a scenario in which the initial project would be 100% in monitored continuous auger piles, as shown in Table 6.

Indicator	Initial project	Modified design	Idealized project
Total length drilled (m)	15.008,00	14.625,47	14.328,90
Concrete volume (m^3)	14.371,83	13.746,85	13.444,47
Steel (kg)	710.502,87	519.047,74	490.050,28
Execution time (h)	2.237,36	838,22	641,16

Table 6. Comparison between the initial, modified and idealized project. Prepared by the author (2024).

Figure 1 below graphically demonstrates the percentage reduction of each indicator shown in Table 6.

CONCLUSION

In view of the results obtained and the analysis carried out, it can be seen that the change in the methodology used to execute the foundations has brought substantial gains to the project, especially in terms of time and input savings, factors directly linked to the financial success of a project.

It is believed that the project could have been optimized to a much greater extent, even in relation to the figures of the idealized project presented, if a load test campaign had been carried out while the project was still being drawn up. This reinforces the need for greater financial investment in the foundation design phase, which tends to guarantee greater technical reliability for the project and an obvious reduction in time and cost.

The fact is that most of the semi-empirical methods for geotechnical design were developed based on static load tests in regions that are not very representative of Brazil's geological diversity, generating possible conservatism and distance from the reality of the actual load capacity of piles. In addition, advances in hydraulic drill technology have made it possible to drill in very competent soils that were considered impenetrable years ago, a circumstance that makes the application of executive methodologies feasible in more works.

It can be said that this article contributes to the geotechnical community by helping to dimension the geotechnical load capacity of deep foundations, since there is no published work in this niche for the region of Canaã dos Carajás-PA. In addition, it reinforces the possibility of saving costs and time by carrying out a load test campaign before issuing the executive design of the foundations for any project.

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