Numerical Analysis of an Embedded Pile in Fractured Rock

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Abstract. The determination of the load capacity of deep foundation elements is complex and requires the definition of a good physical and mathematical model that expresses an adequate approximation of the installation process, interaction mechanisms and the failure mode. In particular, piles embedded in rocks add difficulty to predictions due to scarcity of specimen tests of the rock to define appropriate stress-strain relations, and information about rock integrity in field. To overcome these issues, the Brazilian national practice of deep foundations uses semi-empirical methods to predict load capacity validated with loading tests. In this context, the present work presents a load capacity analysis of a deep foundation element embedded in rock using finite element approach which is directly compared with axial compression static loading test, and semi-empirical predictions. The pile was modeled considering elastic behavior, soil layers and fractured rocks were modeled considering the elastoplastic model of Mohr Coulomb. Interaction mechanisms were modeled considering a normal rigid contact behavior and penalty for tangential behavior. Results of two analyzes were presented in the paper: an analysis defined considering a set of parameters derived based on in situ investigation (Standard Penetration Test - SPT and Cone test - CPT) and; a parametric analysis to investigate the influence of the interaction coefficient. The results presented contribute to the understanding of the behavior of embedded piles in situ and aims to help in the definition of suitable models for prediction in future works.

Keywords: interaction mechanisms, in situ investigation, load capacity tests.

1 Introduction

According to Hachich et al. [1], the determination of the foundation type depends on the analysis of a series of factors, the geological and geotechnical studies of the subsoil can be considered the most relevant, because, in addition to the soil having an extremely variable nature in terms of composition and behavior when submitted to loads, it is commonly the least resistant material of the foundation element, therefore, the load capacity is restricted to the soil-structure interaction.

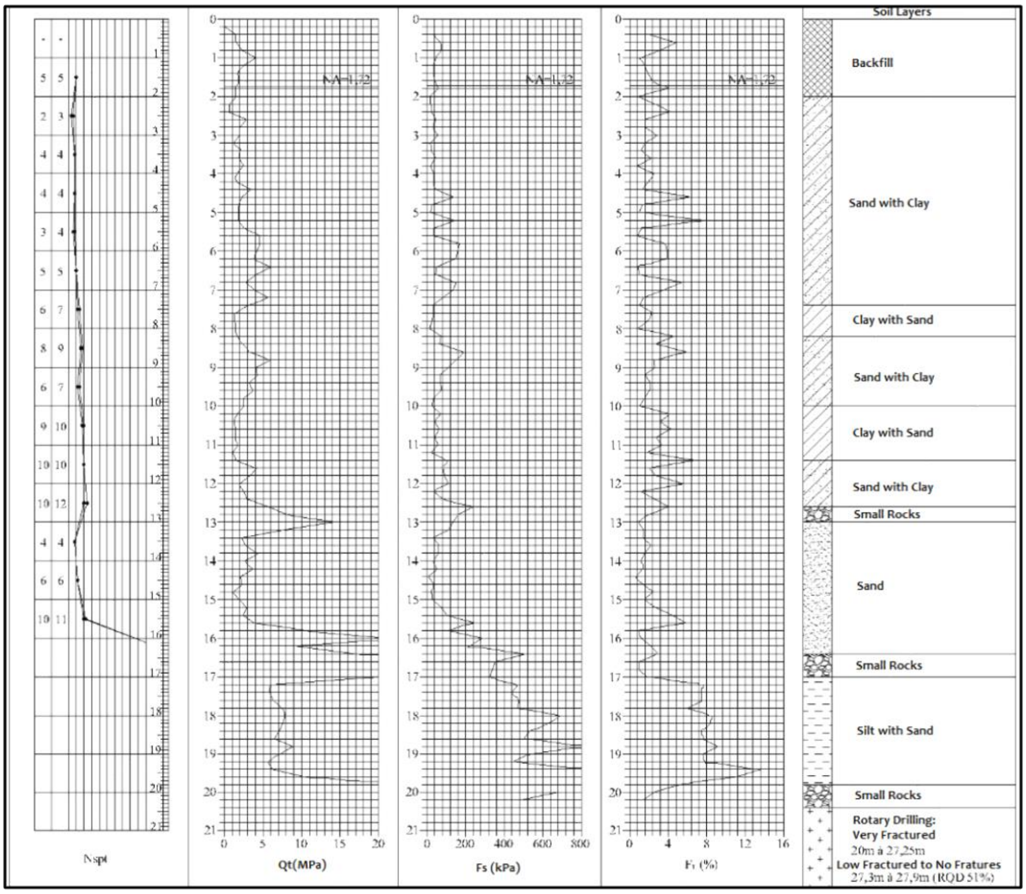
The analyzes are even more complex when the foundations are embedded and/or supported in rock mass, since, unlike soils, its behavior is influenced not only by his constitutive material, but also by the presence of discontinuities and flaws in its internal structure (Goodman [2]). However, in Brazil, there are no specific codes for laying foundations in rocks, for this reason, the only information available is usually the one obtained based on rotary drilling, with no additional tests being carried out to verify the rock mass properties. With the advent of technology in the 1950s, the use of the Finite Element Method (FEM), which consists of discretizing the problem domain into smaller finite-dimensional elements, became one of the options for determining the load capacity. The FEM gives assessment to a detailed and realistic values of tension, deformation and resistance, who must take the proper precautions to ensure results consistent with reality.

In this context, the present work presents the analysis of the axial load capacity of a deep foundation element, a continuous helix pile embedded in rock. The result of the prediction obtained through the numerical modeling was directly confronted with the experimental value obtained from the static load compression test, in order to identify the best behavior representation of the evaluated pile.

2 GEOTECHNICAL STUDY OF THE SITE

The study site is located in Itapema, municipality of the State of Santa Catarina, Brazil. It is a coastal region, characterized by fluvial and marine deposits, formed by thick layers of soft clays interspersed with sand lens, which do not provide adequate support for shallow foundations, were the use of deep foundations is indicated.

Figure 1 presents the geological profile of the subsoil obtained by in situ tests, a mixed, percussion and rotary drilling (Fig.1a) and cone penetration test CPT (Fig.1b, c and d), showing a profile composed by sandy layers interspersed by clay layers along the 20 m depth investigation. N_{spt} index from 5 to 12 were characterized from surface to 16 meters characterizing the impenetrable from this depth. However, the rotary test was conducted from 20 to 27.9 meters with sample recovery only from 27.3 to 27.9 meters with a recovery index of 51% (Itapema Granite with 51% RQD).



(a) (b) (c) (d) (e)

Figure 1. Geological-geotechnical profile of the study Site (a) Nspt profile; (b), (c) and (d) cone test results of tip resistance, Qt, Friction Resistance, Fs, and Friction Ration, Fr; (e) geological description.

Cone tip resistance (Qt) were shown in Fig. 1(b) increasing from 2 to 20MPa from surface to 16m depth, in accordance with the percussion test. However the CPT sounding progresses from 17 to 20 meters with a resistance around 7 MPa until it is interrupted around 20m with Qt greater than 20MPa.

Based on the set of investigations presented, it was considered that the granite rock starts at 27.3m

3 Methodology

The present study makes a direct comparison between three types of results, the load test carried out on site, the extrapolation of the load test using the Van Der Venn method (1953) and the numerical model carried out with the aid of finite element software (Abaqus).

3.1 Static Load Test

A compression load test was carried out on a continuous helix pile, with a diameter of 0.60 m and a length of 26.07 m. The pile cropping level was -1.00 m in relation to the level of the natural ground, with its tip positioned at a depth of -27.07 m. The procedure was carried out in accordance with the precepts indicated in NBR 16903 [3]. The test result is shown in Fig.2.

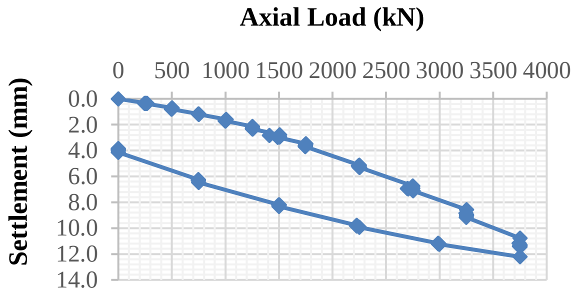


Figure 2. Load Test Results

3.2 Van der Venn Method (1953)

According to Hachich et al. [1], the extrapolation criterion proposed by Van der Veen in 1953 is probably the most used in Brazil. The method consists of associating the load-settlement curve with an exponential function, according to eq. (1).

$$P = P_{\text{ult}} \cdot (1-e^{-\alpha \cdot \rho}) \tag{1}$$

where P is the load applied on the top of the pile, P_{ult} is the ultimate or rupture load, α is the coefficient that defines the shape of the curve (in units of mm-1) and ρ is the settlement corresponding to the applied load.

Rewriting eq. (1) based on the logarithmic properties, the equation of a straight line is obtained, described by eq. (2).

$$\alpha \cdot \rho + \ln(1-P/P_{\text{ult}}) = 0 \tag{2}$$

The coefficient that defines the shape of the curve (α) and the ultimate charge (P_{ult}) are constants determined through trials, where values for P_{ult} are adopted and the respective values of $-\ln(1-P/P_{ult})$ are calculated. The values obtained are plotted on a graph as a function of the settlement (ρ), the resulting graph that is closest to a straight line will indicate the sought value of P_{ult} and the value of α will be given by the slope of the line, as indicated in Fig. 3 (Cintra et al. [4]).

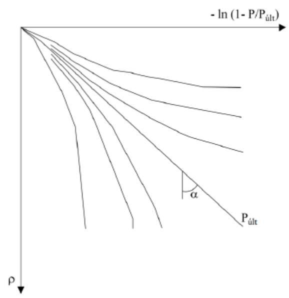


Figure 3. Graphical Solution for the Van der Veen Method (1953)

3.3 Numerical Modeling

Abaqus is a finite element software that allows the numerical behavior simulation of different materials. It is frequently used in several areas of engineering, showing ascending use in Geotechnics, due to its great versatility (Lautenschläger [5]). Several applications of the program for solving geotechnical problems can be found in Helwany [6].

One of the main requirements for performing numerical analyzes is the choice of the constitutive model that best represents the real behavior of the materials that involve the problem. According to De Vos and Wenham (2005) apud Faro [7], the constitutive models are divided into two groups: linear elastic behavior (or non-linear elastic) and plastic behavior, which include the models of Tresca, Von Mises, Mohr - Coulomb, Drucker-Prager and Cam-Clay.

For the present research, the linear elastic constitutive model was used to characterize the foundation element, and for the soil and rock layers, the Mohr-Coulomb elastoplastic model was used. The input data, for the reinforced concrete pile with fck = 40.0 MPa, are presented in Tab.1.

Table 1. Concrete Parameters of the Piles

Pile	**E (kN/m²)**	**ν**	**γ (kN/m³)**
ET.01 e ET.02	35.000.000 [1]	0,2 [1]	25,0 [2]

Reference: [1] NBR 6118 [8] e [2] NBR 6120 [9].

According to Dienstmann [10], the Mohr Coulomb model is widely applied due to the familiarity with the rupture criterion and his few data requirement input, which are easily obtained through laboratory tests. The criterion determines the failure envelope through the shear stress (τ) at the imminence of failure, in the failure plane, and is expressed by eq. (3).

$$\tau = c' + \sigma' . tg\ \phi' \tag{3}$$

Where σ′ is the effective stress normal to the shear plane.

As for the parameters, Abaqus requires the Young's modulus (E) and Poisson's ratio (ν) to represent the elastic behavior and to define the plasticity criteria, the effective friction angle (ϕ′), the effective cohesion (c′) and the dilation angle (ψ). The input data for each of the layers is shown in Tab. 2.

Table 2. Assumed Geotechnical Parameters

Layers	**E (kN/m²)**	$\boldsymbol{v}$	**ϕ′ (°)**	**c′ (kN/m²)**	$\boldsymbol{k}$ **(m/s)**	$\boldsymbol{e}$	**γ (kN/m³)**
Silt, with Sand and Clay, medium dense (Above Water)	11.000 [3]	0,33 [3]	28,5 [3]	1,00	10^{-7} [4]	0,75 [5]	14,50 [2]
Silt, with Sand and Clay, medium dense (Under Water)	11.000 [3]	0,33 [3]	28,5 [3]	1,00	10^{-7} [4]	0,75 [5]	16,50 [2]
Sand with Clay, Loose	19.000 [1]	0,30 [1]	31,0 [3]	1,00	10^{-5} [1]	0,80 [1]	19,25 [2]
Coarse Sand, with Clay, Loose to Medium Dense	19.000 [1]	0,30 [1]	31,0 [3]	1,00	10^{-3} [1]	0,80 [1]	19,25 [2]
Clay, Medium	10.000 [1]	0,35 [1]	26,0 [10]	36,77 [9]	10^{-8} [1]	0,60 [1]	15,25 [2]
Clay with sand, Medium to Stiff	10.000 [1]	0,25 [3]	26,0 [10]	73,55 [9]	10^{-8} [1]	0,60 [1]	16,00 [2]
Clay with Silt, Soft to Medium	9000 [19]	0,35 [1]	26,0 [10]	36,77 [9]	10^{-6} [1]	1,15 [1]	16,00 [2]
Silt with Sand, Very Dense	11.000 [3]	0,33 [3]	32,5 [3]	1,00	10^{-7} [4]	0,75 [5]	16,50 [2]
Itapema Granite, Very Fractured	52.700.000 [2]	0,25 [2]	50,0 [7]	12700 [7]	10^{-9} [6]	0,25	28,50 [8]

Reference: [1] Das [11], [2] Coduto [12], [3] Bowles [13], [4] Pinto [14], [5] Fernandes [15], [6] Goodman [2], [7] Rocha (1981) apud Lins [16], [8] NBR 6120 [9], [9] Marangon [17], [10] Oliveira [18], [19] Teixeira e Godoy (1996) apud Veloso e Lopes [20].

For numerical modeling, different meshes were used in the modeling of the pile and soil layers, due to the fact that they have geometry and materials with different characteristics. In the pile, a mesh with four-node bilinear axisymmetric quadrilateral elements "CAX4R" was used, with reduced integration and distortion control. For the soil layers, in turn, a mesh with axisymmetric quadrilateral elements "CAX4P", with four nodes, with displacement and pore pressure, both bilinear, was considered.

In order to reduce the calculation time of the simulations, linear interpolation and reduced integration (a single point of integration) were chosen. In order to maintain the quality of the results, even after the simplifications performed, the meshes were refined in the regions around the pile, where the greatest stresses and displacements are expected, as exemplified in Fig. 4.

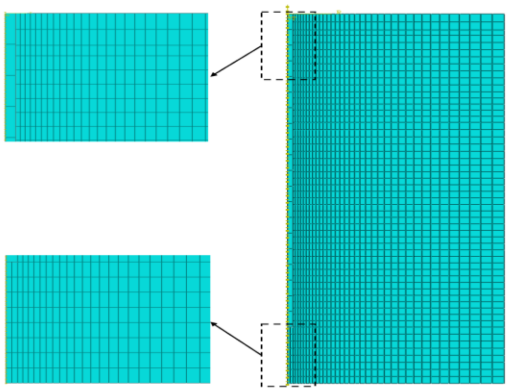


Figure 4. Adopted meshes with refinement

For the soil-pile interaction, in the numerical modeling performed, the Surface-Surface (S-S) discretization technique was used, defining one surface as "master" and another as "slave". The friction method adopted was the "penalty" method, with a starting value of 0.3, which is in good agreement with the literature.

4 Results

4.1 Static Load Test Interpretation

The analyzed load-settlement curve doesn't show a clear rupture, therefore, it was necessary an extrapolation of by the physical rupture criterion, proposed by Van der Veen (1953), in order to determinate the load capacity through the conventional rupture criterion suggested by NBR 6122 [19].

To use of the Van der Veen method, the graphs of $- \ln \cdot (1\text{-}P/P_{ult})$ were plotted as a function of the settlements measured in the load test, assuming values for the rupture load (P_{ult}) as described in section 3.2. According to Cintra et al. (2013), when R^2 is sufficiently close to 1.00, it can be said that the extrapolated curve is in accordance with the experimental curve. Thus, it appears that the best straight line presents R^2 equal to 0.9996, with a rupture load of 6,500 kN. After carrying out the necessary unit conversions, an angular coefficient of 0.09028 mm-1 is obtained and the extrapolation becomes a function of Eq. (4).

$$P = 6.500 \cdot (1\text{-}e^{(-0,09028 \cdot \rho)}) \tag{4}$$

The extrapolated load-settlement curve and the straight line obtained by the NBR 6122 [19] criterion are show in Fig. 5. The load capacity is determined by the intersection of the straight line with the extrapolated curve, resulting in 6,247.96 kN (66.61% greater than the tested load of 3,750 kN).

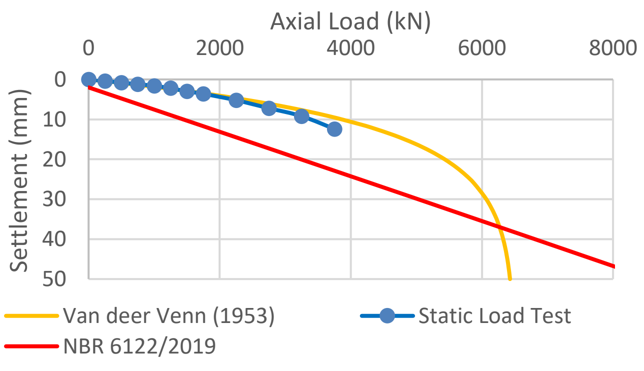
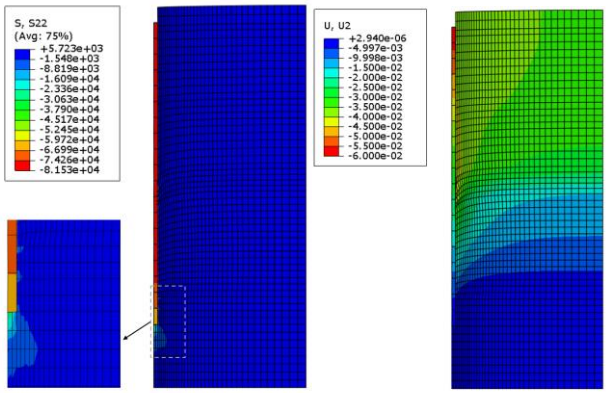


Figure 5. Rupture for NBR 6122/2019 Criterion

4.2 Numerical Model

With the results from the numerical simulations, it is possible to obtain, in addition to the axial load capacity, information about stresses and strains. Fig. 6(a) illustrates the regions of the pile-soil system that suffer greater stress in terms of vertical stress distributions. It is observed that the maximum concentration occurs in almost the entire length of the pile, reaching values around 81.53 MPa. As for soil stresses, the concentration is only noted in the region just below the tip of the foundation element, with a maximum value of 23.36 MPa.



(a) **(b)**

Figure 6: Soil-Pile System: (a) Vertical Stress (kPa); (b) Displacement (m)

It can be verified through Fig. 6(b) that the vertical displacements tend to decrease with depth. The largest displacements occur at the top of the pile, with a corresponding value of 60.0 mm in the direction of the applied load. The reaction force and the vertical displacement, to design the load-settlement curve of the numerical model, were obtained at two nodes located at the top of the pile.

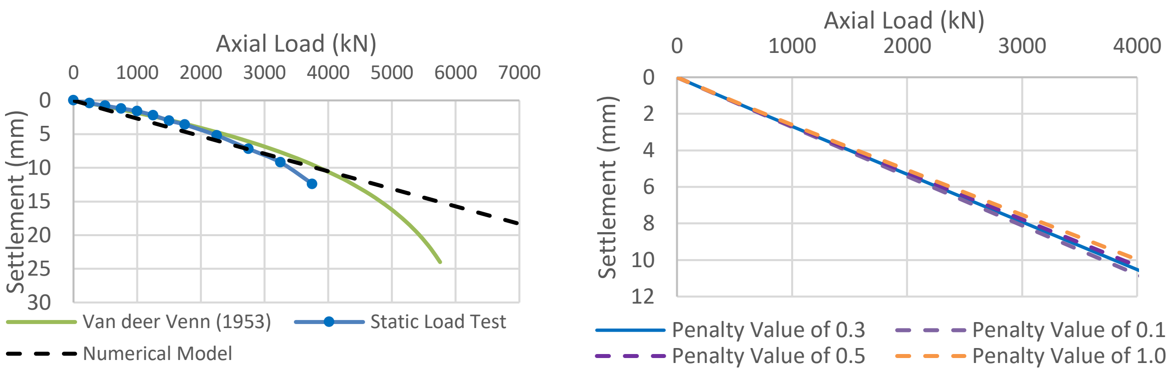


Figure 7. Load-settlement curves (a) static load test, numerical modeling and extrapolation by Van der Venn Method (1953) (b) parametric evaluation of the interface friction value

Note, based on Fig.7a, that the curve obtained in the simulation presents an initial behavior similar to that of the load-settlement curve given by the static load test and by its extrapolation through Van der Venn Method (1953), however the numerical behavior curve continues to grow continuously, not showing asymptotic behavior. Demonstrating a pure elastic shortening behavior of the pile. In order to compare prediction and load test values, it can be observed that for the maximum displacement of the load test, the numerical prediction characterizes a maximum axial load of 5,114.84 kN, 36.40% greater than the tested load of 3,750 kN.

Without changing the estimated soil parameters, it was investigated in a preliminary way if the interface parameter could result in a change of asymptotic behavior in the modeling. Three complementary analyzes were carried out, changing the penalty value of the interface to 0.1, 0.5 and 1.0, the results are plotted on the Fig 7b. As we can be seen, changing the interface parameter smoothly changes the prediction not controlling or creating inflection, even for the low-parameter simulation, which simulates an almost smooth surface (0.1). In this sense, the results indicate that the simulated behavior is mostly an elastic response of the pile.

5 Conclusion

The present work aimed to analyze the axial load capacity of a deep foundation element, continuous helix pile, embedded in rock. Numerical simulations were performed using the Finite Element Method in order to model the soil-pile interaction processes and characterize the stresses and strains mobilized. The comparative analysis of the results took place from the compression static load test.

In summary, numerical simulation presents an initial displacement strength behavior similar to that of the load-settlement curve given by the static load test and by its extrapolation through Van der Venn Method (1953), however the numerical result continues to grow continuously, not showing asymptotic behavior, demonstrating a pure elastic shortening behavior of the pile. For future works, it is recommended that soil and rock parameters would be revisited with sample collection and laboratory tests, also that similar piles be instrumented prior to load-tests, so that a more accurate investigation of load transmission can be evaluated.

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