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Geotechnical field investigations to measure soil resistance on the Boa Vista/RR Western Bypass Highway and solution to embankments on soft soils

Investigações geotécnicas de campo para medição das resistências de solos na Rodovia do Contorno Oeste de Boa Vista/RR e solução aos aterros sobre solos moles

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ABSTRACT
Among other challenges, landfills on soft soils have been inspiring geotechnical engineers, designers and executors to seek challenging solutions. On this topic, Professor Márcio Almeida, one of the greatest experts on the subject, mentions that these challenges have already resulted in successes and memorable ruptures in works based on soft soils. In general, soft soils are considered to be materials with high plasticity, in some cases rich in organic matter, with Nspt ≤ 4. Therefore, brief considerations on soft soils are presented, but also results with analyzes and discussions of geotechnical studies carried out on the Highway West Contour of Boa Vista (BR 174), in the State of Roraima, aiming to determine soil resistance through in situ Vane Tests, in accordance with ABNT NBR 10905: 1989 (MB-3122). The studies consisted of carrying out "Geotechnical Field Investigations" in sections comprising 11 stakes of the aforementioned Highway. Considering the results, it was found that the soft soil layers in the studied sections have "average" thicknesses that vary from section to section, with minimums close to 2.0 meters and maximums exceeding 6.0 meters. Furthermore, the existing embankments of the old runway, located in the sections studied, indicate the presence of soils with low bearing capacity under the body of the embankments. In addition to the studies, solutions are proposed for landfills on soils with low bearing capacity.

Keywords: Contorno Oeste da Boa Vista Highway, Landfill, Soft soil, Resistance.

INTRODUCTION

This work presents the results of geotechnical studies carried out on Rodovia do Contorno Oeste de Boa Vista, state of Roraima (RR), with a view to determining soil resistance through "in situ" vane tests (Vane Test), in accordance with with ABNT NBR 10905: 1989

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(MB-3122). Additionally, to support the analysis of reed tests and also to better understand the geotechnical properties of materials for the embankment project on soft soils, tests were carried out with a Light Dynamic Penetrometer (PDL) in accordance with the recommendations of DIM 4094 and ISSMFE, 1989. , and in compliance with the procedure of the former National Department of Roads and Highways (DNER-PRO 381/98) Shelby sample collection, layer thickness determination, soil characterization and resistance tests. However, this work presents and analyzes the "in situ" reed tests carried out in the area in question.

INITIAL CONSIDERATIONS

In developing this work, we sought a broad bibliographical survey of embankments on soft soils, but also information that could more faithfully represent the geotechnical knowledge of the region studied.

Soft soils are considered to be materials rich in organic matter, with high plasticity and Nspt ≤ 4, organic soil deposits, very soft sand, peat or hydromorphic soils, in general, likely to occur in low-lying floodplain areas; mangroves and swamps; floodplains of low hydraulic gradient rivers; ancient watercourse beds; plains of marine or lacustrine sedimentation. They have physical properties that can vary, resulting in changes in behavior within the same deposit. Characteristics include high compressibility, low resistance, low permeability and low consistency. The characterization of soft soil deposits aims to guide and provide support for road projects in order to avoid future burdens such as excessive deformations, cracks and slope failures.

In general, landfills on soft soils are defined in three classes: Class I, II and III (DNER 381, 1998), according to specific characteristics.

IN SITU REED TEST (VANE TEST)

Its purpose is to measure the undrained shear strength of purely cohesive soils. In Brazil, the standard that governs the test is ABNT NBR 10905:1998.

According to Almeida (1996), the "in situ" vane test is the most used to determine the undrained resistance Cu of soft soil, consisting of the constant rotation of 6° per minute of a cruciform vane at pre-defined depths. The measurement of torque versus rotation allows the determination of Cu values. Table 1 presents the consistency states for soils depending on the penetration resistance index "N" obtained in the SPT tests, based on Annex "A" of NBR 6484/2020.

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Table 1. Soil consistency status (ABNT NBR 6484: 2020).

Ground	N	Designation
Clays and clayey silts	≤ 2	Too soft
	3 to 5	Soft
	6 to 10	Average
	11 to 19	Tough
	20 to 30	Very stiff
	> 30	Lasts

To determine the undrained soil resistance (Cu), Equation 1 is used:

$$Cu = \left[\frac{T}{\pi.\left(D^3 + \frac{D^3}{6} \right)} \right] . \frac{1}{100} \tag{1}$$

Where: Cu = Undrained soil resistance, in kPa; T = Maximum ground torque in kgf.m; D = Reed diameter; π = 3.1416.

SENSITIVITY OF CLAYS

The resistance of clays depends on the arrangement between the grains and the void content in which it is found. The consistency after handling (dented) may be lower than in the natural state (undeformed). This phenomenon occurs differently and was called clay sensitivity.

The sensitivity can be clearly visualized through two simple compression tests. The first with the sample in its natural state. The second, with a test specimen made with the same soil after complete remolding, but with the same void ratio. To determine the clay sensitivity, Equation 2 is used:

$$S = \frac{Resistência\ no\ estado\ indeformado}{Resistência\ no\ estado\ amolgado} \qquad \text{(two)}$$

Regarding sensitivity, clays are classified according to Table 2. It should be noted that the higher the sensitivity level, the lower the soil's support capacity.

Table 2. Classification of clays according to sensitivity.

Sensitivity	Classification	
1	Insensitive	
1 to 2	Low sensitivity	
2 to 4	Medium sensitivity	
4 to 8	Sensitive	
> 8	Ultra-sensitive	

LIGHT DYNAMIC PENETROMETER TEST (DPL)

The DPL, specified in the International Reference for Test Procedures for Dynamic Sounding (DP) (ISSMFE, 1989), is a small manual field equipment designed for use in drillings with a maximum depth of 21 m. Survey suitable for small to medium depth projects, complementing and correlating with other studies. It is ideal for use in difficult to access places such as: floodplains, embankments and closed spaces. With good penetrability in soft/soft and moderately hard soils, it is also possible to use it in layers of moderately compacted sand and gravel. The test provides lateral friction and tip resistance, allowing a slight tactile-visual assessment of the soil through grooves present in the rods and identification of the water level.

The DPL aims to characterize and determine the resistance to penetration of soil deposits with low bearing capacity. This test helps to determine the resistance of soils in areas where the Palheta test becomes technically unfeasible, such as: in granular soils, unsaturated soils and soils with high permeability. Figures 1 and 2 show the components of the Light Dynamic Penetrometer (DPL) equipment.

Figure 1. Components for the DPL assay.

Figure 2. DPL tip for ground penetration.

CHARACTERIZATION OF THE MUNICIPALITY OF BOA VISTA

Boa Vista (Latitude: 2° 49' 10" North and Longitude: 60° 40' 17" West) is a Brazilian municipality, capital of the state of Roraima, northern region of the country. It is located on the right bank of the Rio Branco and is located at an altitude of 76 meters. It is the northernmost capital of Brazil and the only one located completely north of the Equator, and the furthest from Brasília, the federal capital. According to IBGE, it has an area of 5,687.037 km² and an estimated population in 2019 of 399,213 inhabitants.

SOILS IN THE STUDY AREA

Vale Júnior (2005), mentions that the soils in the state of Roraima are related to their geomorphology. The soils have as striking characteristics low natural fertility, low base

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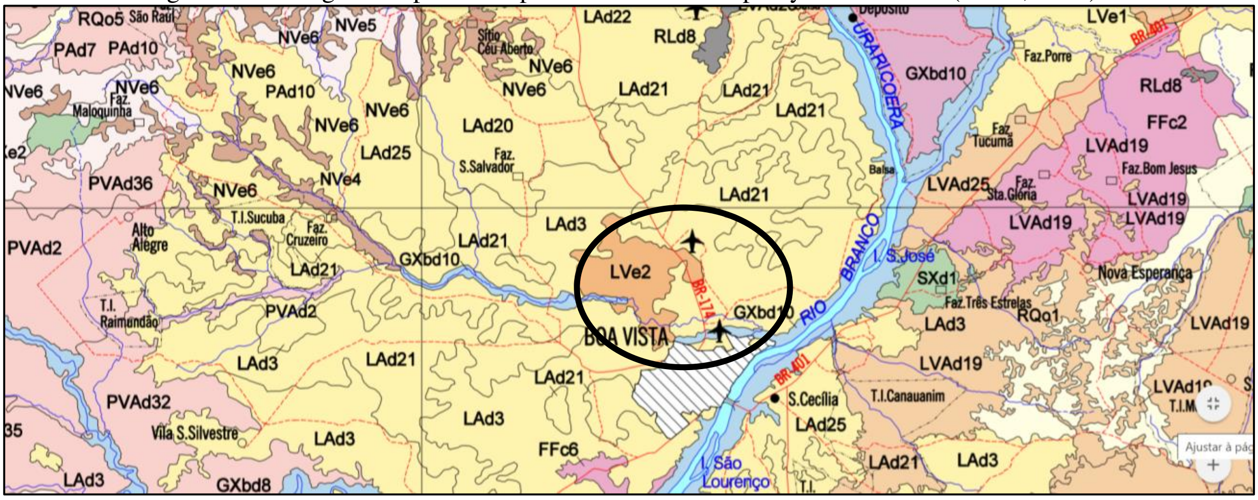
saturation and high aluminum saturation. Figure 3 shows a small fragment of the pedological map of the studied region in which predominantly latosol, neosol, plinthosoil and gleisol were found.

Figure 3. Pedological map with emphasis on the Municipality of Boa Vista (IBGE, 2005).



WORK METHODOLOGY

The methodology involved a broad bibliographical survey and results of geotechnical studies carried out in 2008, on the Contorno Oeste de Boa Vista Highway in the State of Roraima and aimed at determining soil resistance through in situ Vane Tests, in accordance with ABNT NBR 10905: 1989 (MB-3122), and the Light Dynamic Penetrometer (DPL) in accordance with recommendations from DIM 4094 and ISSMFE, 1989. The studies consisted of the execution of a program of “Geotechnical Field Investigations” in sections included in stations 181, 183, 186, 188, 378, 380, 382, 384, 425, 427 and 429 of the aforementioned Highway. Figures 4 to 9 show test locations.

Figure 4. View of section 1 (Piles 181 to 188).

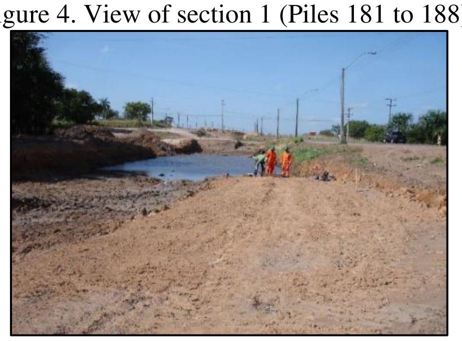


Figure 5. Existence of Buritizeiros indicating the presence of soft soil.

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Figure 6. Existence of soft soil in section 1.

Figure 7. Soil profile of section 1.

Figure 8. DPL test execution.

Figure 9. Preparation for carrying out the “vane-test”.

PRESENTATION AND ANALYSIS OF RESULTS OF THE SOIL PROFILES STUDIED

In total, 3 (three) sections were studied, as shown in Table 3, which presents the approximate length of each section with the corresponding stakes.

Table 3. Sections where the “Vane-Test” tests were carried out.

Excerpt	Approximate Length (m)	piles
01	160m	181, 183, 186 and 188
02	140m	378, 380, 382 and 384
03	140m	425, 427, 429+15

Figures 10, 11 and 12 present the profiles of undrained resistance x depth measured through reed tests for Sections 01, 02 and 03. Then, in Figures 13, 14 and 15, the undrained resistance dented in them depths. In addition, Figures 16, 17 and 18 show results of soil sensitivity x depths.

Taking into account the undrained resistance of Section 01 (Figure 10), there is some variability in the measured values, with maximum undrained resistance close to 130 kPa up to a depth of 4.0 meters in piles 181 and 188.

The undrained resistance for Section 02 (Figure 11), at a depth of 0.50 m, presented values from 39 kPa to 76 kPa with a decrease until close to a depth of 4.5 m, for a subsequent increase until a depth of 6.0 m in the cuttings 378; 380 and 384. However, in pile 382 the soil

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presented resistance below that found in the other three piles. It is possible that the higher values in the 0.5m layer are related to the drying out of the surface layer, which does not represent reliability in the resistance at this depth, as these layers are subject to variation in resistance during periods of drying and drought.

When analyzing section 03 (Figure 12), there is a higher resistance in the initial layer, as seen in section 02, with a decrease in the lower layers. For piles 425 and 429 + 15, an increase in undrained resistance can be seen from an approximate depth of 3.5m. This fact was not observed in pile 427, where the resistance was maintained or decreased up to a depth of 6.0 m.

Observing the undrained dented resistance for section 01 (Figure 13) there is a certain initial trend up to a depth of 2.0 m. Then there is a considerable gain in resistance in pile 181, with values close to 130 kPa. For pile 183, a certain variability is observed with an increasing increase from 4.5 m to a depth of 6.5 m. It is possible that the great variability in resistance of piles 181 and 183 is due to the fact that the soil in the studied area is composed of sedimentary soil, which is transported soil, and is subject to considerable variability in resistance.

In relation to the undrained resistance dented in section 02 (Figure 14), there is similar behavior for piles 380; 382 and 384 up to a depth of 2.5 m with maximum resistance less than 20 kPa. There is also a gain in resistance greater than 70 kPa for piles 378 and 384. This section is generally characterized by soil that is considered soft, as most of the piles do not exceed 50 kPa of resistance.

Taking into account the dented undrained resistance for Section 03 (Figure 15), there is some variability in the initial measured values, however from a depth of 1.0 m there is a certain tendency with resistances reaching a maximum of approximately 23 kPa, with except for pile 429+15, which has a considerable gain in resistance from 4.5 m, reaching close to 71 kPa. The average resistance of the three piles is 40.67 kPa, which is characterized as soft soil.

Figure 10. “Vane-Test” for section 01.

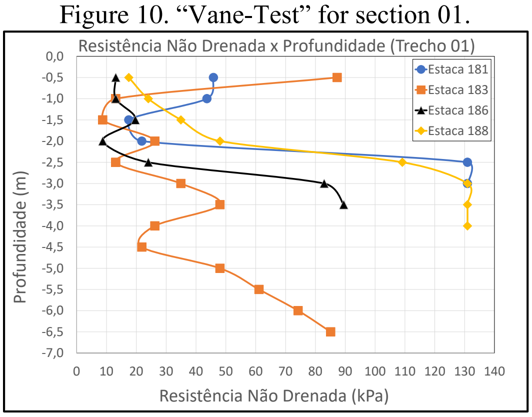


Figure 11. “Vane-Test” for section 02.

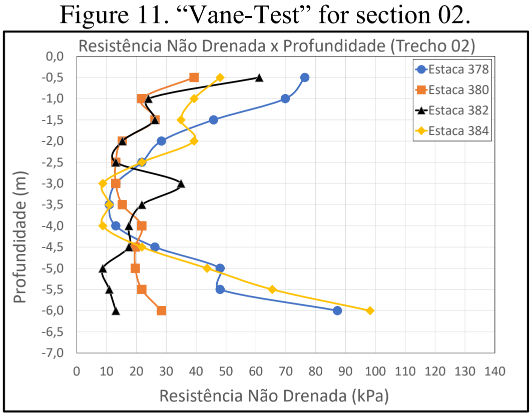


Figure 12. "Vane-Test" for section 03.

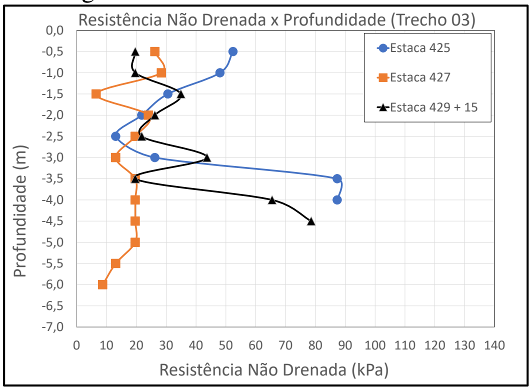


Figure 13. Undrained dented resistance x depth for section 01.

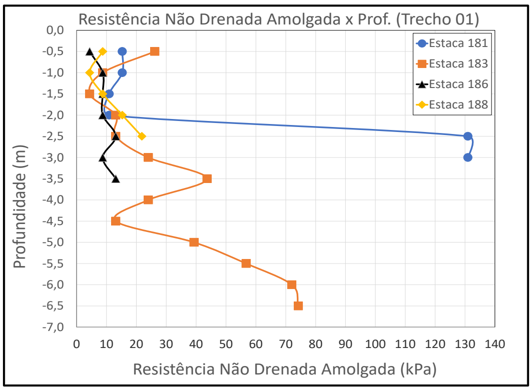


Figure 14. Undrained dented resistance x depth for section 02.

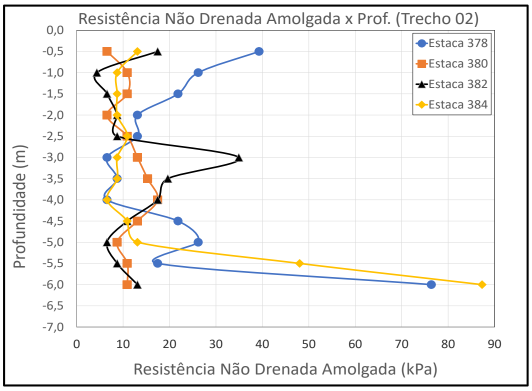
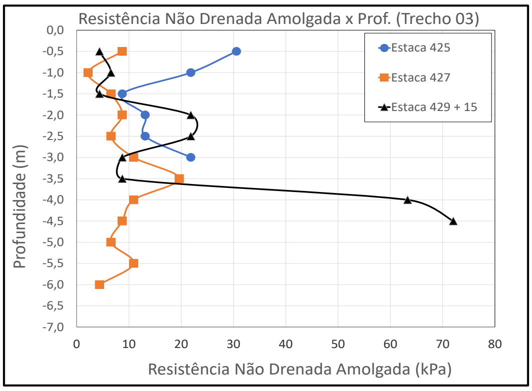


Figure 15. Undrained dented resistance x depth for section 03.



In Section 01 (Figure 16), there is a certain initial tendency for piles 181, 183 and 186 to a depth of 2.5 m, with maximum resistances of 3 kPa, and a considerable increase in sensitivity for pile 186, reaching the value approximately 9.5 kPa, indicating ultra-sensitivity of the soil, with a subsequent decrease close to 7 kPa at a depth of 3.5 m and sensitive behavior. Piling 188 does not follow the same layout as the others, as it has a variability of increases and decreases in sensitivity over depths of 0.5 m and 2.5 m.

In Section 02 (Figure 17) the sensitivity presented values between 3.5 and 5.5 up to 1.5 m for piles 382 and 384 with behavior between medium sensitivity and sensitive. Between 2 and 6 m depth, the soil behaved with indicative values up to medium sensitivity.

Figure 16. Sensitivity x depth excerpt 01.

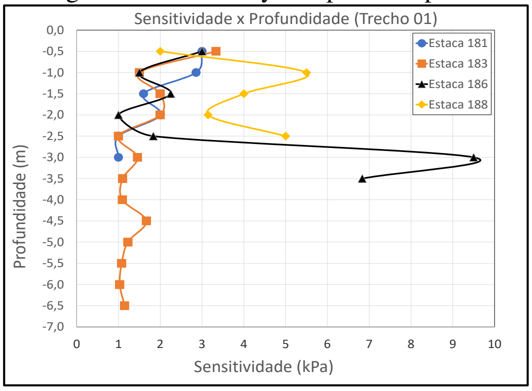
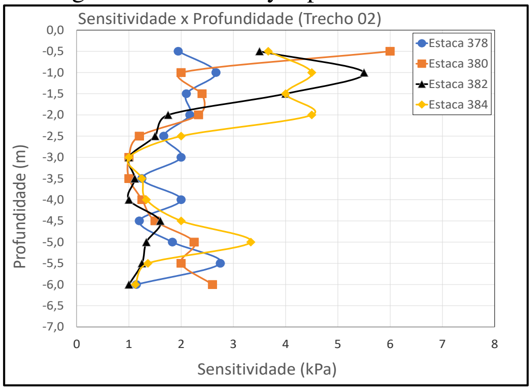
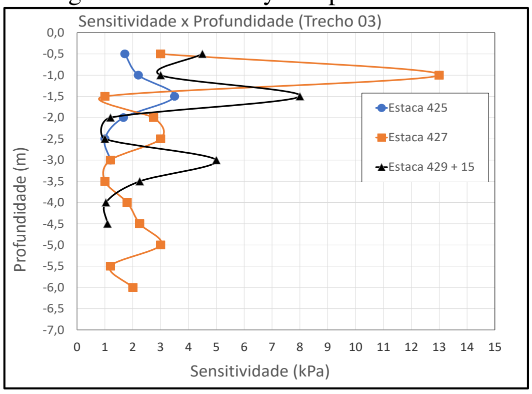


Figure 17. Sensitivity x prof. section 02.



When analyzing the sensitivity for Section 03 (Figure 18), there is a high value of up to 1.0 m for pile 427, reaching a resistance close to 5.2 times greater than the average resistance of piles 425 and 429+ 15. From a depth of 2m, the maximum sensitivity value was 5kPa, indicating even sensitive behavior.

Figure 18. Sensitivity x depth section 03.



FINAL CONSIDERATIONS / RECOMMENDATIONS

NBR 6484 Soil – Simple recognition surveys with SPT – Test method, very soft to soft soil is considered when NSPT ≤ 5, however in the work in question, soft soil was considered when NSPT ≤ 4 and resistances up to 50 kPa. In this context, the main considerations of the study carried out are presented.

SOFT SOIL LAYER WITH THICKNESS OF LESS THAN THREE METERS

With the results obtained, it is concluded that the soils studied, in general, have low support capacity, and that to solve the problems with regard to the densification and stability of embankments to be built in areas where compressible soils occur, it is necessary to remove them.

Complete removal is recommended, that is, the base of removal must be located on material with good support qualities, avoiding remnants of compressible soil that could cause future problems.

In order to stabilize possible pockets of remaining soft soil, it is recommended to lay a layer of rachão stone (stone material of varying sizes, generally up to 20 cm in diameter).

Another important recommendation is the implementation of a drainage system of the drainage trench type (1.50 m deep x 0.50 m wide) associated with the drainage mattress (0.40 m thick), at the base of the landfill, where in addition to the main function (draining) it effectively contributes to reducing differential settlements and increasing resistance by eliminating water. It also reduces capillary rise caused in landfills compacted with silt-clay soils.

SOFT SOIL LAYER WITH THICKNESS MORE THAN THREE METERS

For landfills on soils with low bearing capacity and with heights greater than three meters, cleaning and surface leveling of the area is recommended, using light equipment or manually.

Over the regularized layer, it is recommended to lay a layer of rachão stone until stabilization. On the already stabilized layer, it is recommended to implement a drainage system consisting of a drainage mattress (0.40 m thick), using as a filter a non-woven geotextile with a minimum tensile strength of 31 kN/m and a drainage trench on the perimeters (1.50m deep x 0.50m wide), associated with the drainage mattress. Another layer of non-woven geotextile with a minimum tensile strength of 31 kN/m must be placed on top of this drainage mattress before laying the compacted landfill.

Depending on the need, in relation to the execution time of the compacted landfill, vertical drains (geodrains) may be executed with the aim of accelerating settlements and/or the use of geogrids to minimize differential settlements. Furthermore, it should be noted that there are other solutions that can be analyzed depending on particular situations. Such solutions include the use of light embankments with expanded polystyrene (EPS), balancing berms, temporary overloading, construction in stages, among others that are technically possible and economically viable.

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The Catholic University of Brasília (UCB), Centro Universitário de Brasília (UniCEUB) and Reforsolo Engenharia, whose important contributions made this work possible.

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