Integrated Characterization Using SCPTu, CPTu, and Dynamic Penetration Tests for Geotechnical Sustainability

Luis Andrade-Pais[1)], Leonardo Marchiori[2*)], Maria Vitoria Morais[3)], Luis Ferreira Gomes[4)], Antonio Albuquerque[5)]

[1)] GeoBioTec, Civil Engineering and Architecture Department, University of Beira Interior, Calçada Fonte do Lameiro 1 – 6201-001 Covilhã, Portugal, ljap@ubi.pt; ORCID: 0000-0001-5245-2445
[2*)] GeoBioTec, Civil Engineering and Architecture Department, University of Beira Interior, Calçada Fonte do Lameiro 1 – 6201-001 Covilhã, Portugal, leonardo.marchiori@ubi.pt; ORCID: 0000-0001-8825-2162
[3)] GeoBioTec, Civil Engineering and Architecture Department, University of Beira Interior, Calçada Fonte do Lameiro 1 – 6201-001 Covilhã, Portugal, vitoria.morais@ubi.pt; ORCID: 0000-0003-3895-1672
[4)] GeoBioTec, Civil Engineering and Architecture Department, University of Beira Interior, Calçada Fonte do Lameiro 1 – 6201-001 Covilhã, Portugal, lmfg@ubi.pt; ORCID: 0000-0002-2367-373X
[5)] GeoBioTec, Civil Engineering and Architecture Department, University of Beira Interior, Calçada Fonte do Lameiro 1 – 6201-001 Covilhã, Portugal, antonio.albuquerque@ubi.pt; ORCID: 0000-0001-7099-0685

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Abstract

The present study focuses on the geotechnical characterization of a rock mass in Covilhã (Portugal) using in situ testing techniques. The research aims to enhance the understanding of subsurface conditions by integrating multiple geotechnical investigation methods, including the seismic piezocone penetration test with pore pressure measurements (SCPTu), without seismic (CPTu), dynamic penetration tests (DPM and DPSH). These tests provide critical data on soil mechanical resistance, internal structure, and deformation properties, supporting more accurate foundation and infrastructure design decisions. The results indicate that the studied soil primarily consists of sand to silty sand with high and increasing over consolidated ratio and relative density, showing a trend towards cementation. The SCPTu test proved highly effective in assessing initial stiffness and computing deformation moduli, significantly contributing to geotechnical engineering applications. Additionally, DPM and DPSH have provided complementary data, allowing for a more comprehensive evaluation of subsurface variability. The integration of these methods enhanced the reliability of soil classification, ensuring robust datasets for geotechnical analysis and infrastructure development. This research aligns with the themes of in-situ testing for geotechnical development. The findings emphasize the importance of multidisciplinary approaches in advancing infrastructure resilience and sustainability through accurate subsurface characterization. The combination of SCPTu, CPTu, and dynamic penetration tests supports the development of safer and more efficient geotechnical engineering solutions, reinforcing the role of geotechnics in modern civil engineering challenges.

Keywords: *SCPTu, CPTu, DPM, DPSH, in situ testing, geotechnical characterization, infrastructure sustainability*

1. Introduction

Geotechnical characterization of rock masses and soils is essential for foundation engineering and underground construction [1, 2]. Traditionally, laboratory tests have been widely used to determine soil mechanical properties, however, *in situ* tests have provided fast and efficient continuous underground profiles. Among them, CPTu (cone penetration test with pore pressure measurement) is recognized for its reliability and cost-effectiveness, within the advanced version, the SCPTu, which incorporates seismic sensors, allowing for shear wave velocity (V_s) measurements, fundamental for determining soil stiffness [3, 4]. In addition to, dynamic penetration methods such as DPM (dynamic probing medium-weight) and DPSH (dynamic probing super heavy) offer crucial insights into soil resistance and heterogeneity. The integration of DPM and DPSH with CPTu and SCPTu enhances geotechnical characterization by combining penetration resistance data with stiffness and pore pressure measurements.

Robertson (2021) [5] proposed an update to existing methods for estimating the large-strain liquefied or remoulded undrained shear strength in sand-like, clay-like, and transitional soils. This update recognizes that soil behaviour can range between sand-like and clay-like, influencing CPT-based interpretations for large-strain undrained shear strength due to varying drainage conditions during testing. Schnaid et al. (2020) [6] introduced a methodology for soil classification and state parameter evaluation in granular soils using seismic cone penetration data. Their approach employs a two-stage classification system, first categorizing soil types and drainage conditions, followed by assessing soil behaviour based on the ratio between small-strain shear modulus and cone tip resistance. This classification is then integrated with the state parameter model [7] to identify sands that may exhibit strain softening under undrained loading conditions.

The results should provide valuable information on the mechanical resistance, internal structure, and geotechnical properties of the rock mass, allowing for a more accurate assessment of subsurface conditions [8]. The soil classification, influenced by the packing state and stiffness of the rock mass, indicates sand to silty sand, with increasing over consolidated ratio (OCR) and relative density (D_r), tending towards cementation. The combination of SCPTu, CPTu, and dynamic penetration tests enhances the reliability of soil characterization, ensuring robust data for engineering applications.

Thus, this article presents a detailed study on the characterization of a rock mass using SCPTu, CPTu, and dynamic penetration tests (DPM and DPSH) conducted at the Tortosendo Industrial Park, Covilhã. The results are compared with empirical and laboratory methods,

highlighting the advantages and limitations of in situ approaches for soil classification and geotechnical assessment.

2. Materials and Methods

Figure 1 presents a detailed map of the testing site in Tortosendo Industrial Park, Covilhã (Portugal), showing the location of probing points (station 1 to 10).

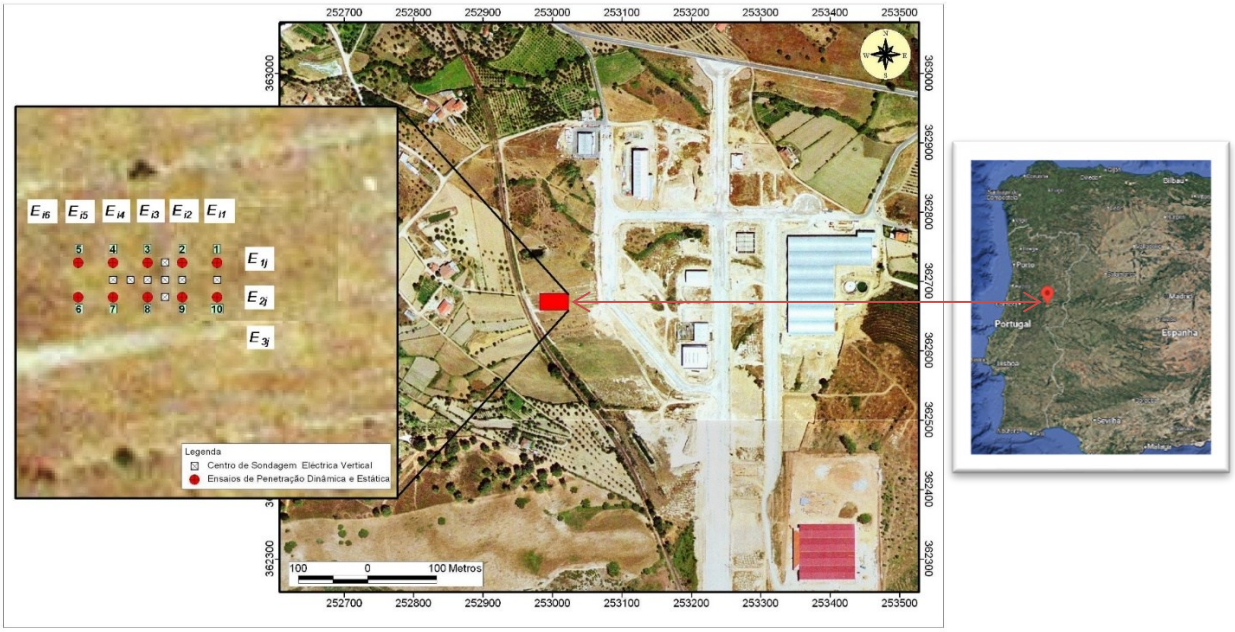


Fig. 1. Testing location (Industrial Zone of Tortosendo, Covilhã - Hayford Gauss, *Datum* Lisboa).

2.1 DPM and DPSH

The DPM and DPSH measure the effort required for a cone to penetrate the soil, providing empirical or semi-empirical correlations with soil resistance and compressibility. The penetration resistance is defined by the number of blows needed to drive the cone a specified distance. The energy per blow depends on the hammer mass, gravity acceleration, and drop height. The dynamic penetration equipment used in this study was acquired from PAGANI TG 63, with technical specifications outlined in [9]. The DPM cone is non-retrievable, whereas the DPSH cone is recoverable. The tests were conducted in 10 different stations across the study area, offering spatially distributed insights into soil resistance and heterogeneity. The PAGANI equipment characteristics are presented in Table 1.

Tab. 1. DPM and DPSH characteristics.

Test	Cone Dimension			Stick Dimension			Pile	
	Diameter (mm)	Angle (°)	Section (cm^2)	Diameter (mm)	Length (m)	Mass (kg)	Mass (kg)	Height (cm)
DPM	35,6	60	10	20	1	2,43	30,0	50
DPSH	50,0	90	20	32	1	6,25	63,5	75

2.2 CPTu and SCPTu

CPT has increasingly been recognized as a valuable in situ tool due to its speed, reliability, cost-effectiveness, and excellent capabilities in defining a continuous soil profile. In this study, the PAGANI TG 63 100 and the PAGANI TG 63 200 were used, with fixed-cone penetrometers capable of applying maximum penetration forces of 110 kN and 220 kN, respectively. The fundamental CPTu parameters include axial force (F_c), required for continuous penetration at a constant speed of 2 cm/s, sleeve friction force (F_s), resistance measured along the sleeve of the cone, pore water pressure (u2), generated due to penetration, and the pore pressure coefficient (B_q), following Equations 1-6:

$$q_c = \frac{F_c}{A_c} \tag{1}$$

$$f_s = \frac{F_s}{A_s} \tag{2}$$

$$R_s = \frac{f_s}{q_c} \tag{3}$$

$$q_t = q_c + u2 \cdot (1 - a) \tag{4}$$

$$a = \frac{A_n}{A_c} = 0.58 \tag{5}$$

$$B_q = \frac{\Delta u}{q_t - \sigma_{v0}} \quad (6)$$

where A_c =10 cm^2 is the cone cross section to correct the q_c, A_s = 150 cm^2 is the exterior area of the sleeve, R_s is friction percentage; A_n is the area of the cell section, besides, Δu = (u-u0) is the generated pore pressure, being u0 the equilibrium pore pressure and total vertical stress (σ_{v0}).

Since the tested materials mainly consist of loose granular soils, and the water table was at a depth of approximately 1.00 m at the time of testing, very fine liquid paraffin was used to saturate the piezocone. In addition to porous ring replacement before each new CPTu test, a specialized device was used to introduce the pre-saturated porous ring into the cone tip, which was then filled with paraffin before reassembly.

3. Results and Discussion

3.1 DPM and DPSH

The dynamic penetration tests (DPM and DPSH) conducted simultaneously suggest that, although DPM is more sensitive, difficulties in maintaining axiality (dependent on the stiffness of the rock mass) affect penetration energy, leading to an overestimation of blow counts. The combined results indicate a significant increase in dynamic tip resistance from 6 m downward, associated with a decrease in the degree of weathering. These results also highlight differential weathering variations within the same plane (spheroidal weathering). The rock mass was excavated by 3 m, and after one year, the tests were repeated at this level. The results revealed a decrease in dynamic tip resistance due to decompression, increased specific alteration area of the mass, and, most notably, changes in σ'_{h0}.

The penetrograms obtained allow the assessment of geomechanical heterogeneity, variation in lithological types, degree of weathering, irregularities, and stiffness transitions leading to bedrock. The penetration resistance results are typically expressed as the number of blows per 10 cm (N10) or 20 cm (N20) of penetration, corresponding to DPM and DPSH, respectively. However, torque measurements to rotate the rods should ideally accompany these values to assess lateral friction, as the latter can lead to overestimated blow counts. Frictional effects increase with depth, the presence of cohesive lithologies, groundwater level, and in situ effective stress (σ'_{h0}). Theoretically, the problem lies in determining the static force required to achieve a constant penetration rate, equivalent to the impact force applied by a falling mass from a known height. The dynamic tip resistance (q_d) is estimated using the semi-empirical Dutch formula for pile driving [9].

$$q_d = \frac{M}{M+M'} \times \frac{M \cdot g \cdot H}{A \cdot e} \quad (7)$$

where M is the pile mass (kg), M' is the total mass of the stick assemble (kg), g is the acceleration (m/s^2), H is the falling height of the pile, A is the area of the base of the cone, and e is the average penetration per blow (m), following 0.1/N10 for DPM and 0.2/N20 for DPSH. The groundwater level (NF) was measured immediately after testing and in the following days, stabilizing at 4.3 m depth.

The highest DPSH penetration resistance was recorded at stations 5 and 4, where N20 > 70 between 3 m and 4 m depth, with deeper stiff layers observed in stations 6 and 7 (h > 11 m). A clear reduction in weather was evident below 6 m depth, leading to increased rock mass stiffness. The DPM showed higher sensitivity due to lower applied energy, yielding consistent results with DPSH up to 3-4 m depth, beyond which rod inclination effects became significant, increasing measured resistance (Fig. 2). To ensure accuracy, vertical alignment must be maintained throughout the test, sometimes requiring pre-drilling. Any deviation causes energy loss and adds lateral friction, affecting results. Groundwater position, lateral friction, and critical depth must be considered, highlighting the need for more sophisticated in situ tests to account for granite residual soil variations.

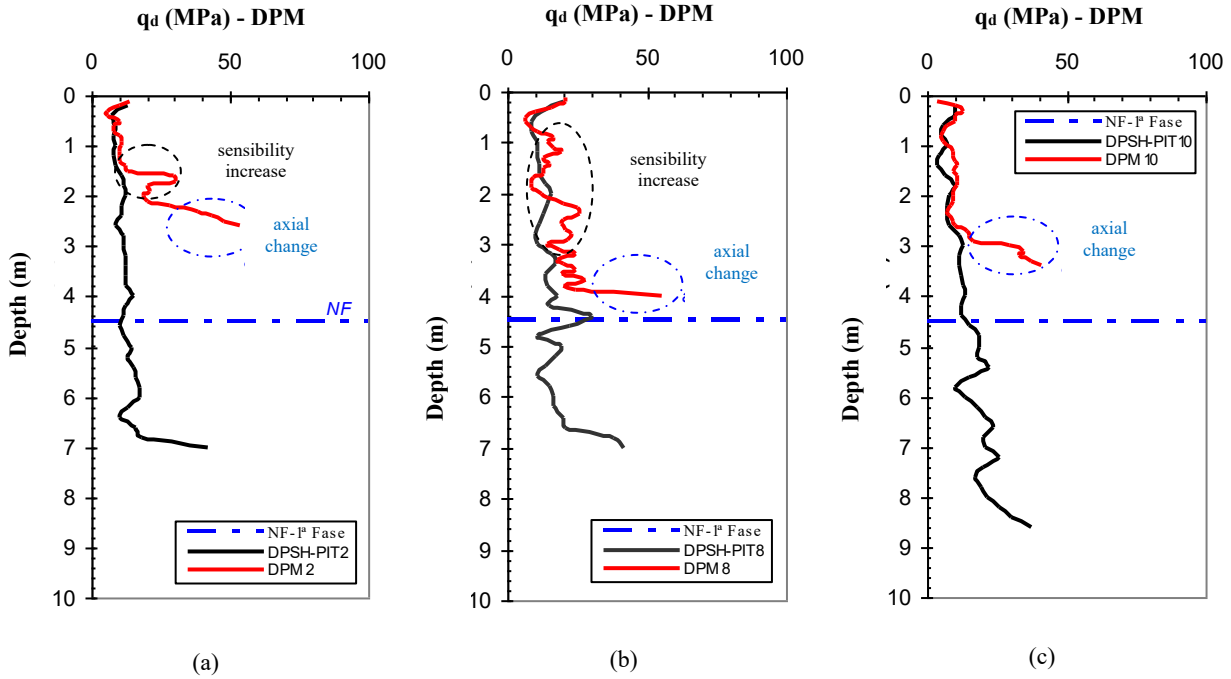


Fig. 2. DPSH and DPM comparison for q_d for station 2 (a) and 8 (b), and 10 (c) with depth.

3.2 CPTu and SCPTu

A total of five CPTu tests were performed, distributed across the defined stations. Fig. 3 presents the results of the tests, referencing the respective stations within the study area. The measured parameters include q_c, f_s, u, and inclination (the angle between the cone tip axis and the defined vertical reference).

Robertson et al. (1986) [10] q_t:B_q projection shows a concentration along the B_q = 0 axis, which aligns with the permeability characteristics of the residual granitic soil. However, there is some dispersion between zones 5 and 9, with a higher concentration in zones 8 and 9. The q_t:R_f projection confirms this dispersion, as R_f parameter is not highly sensitive to structural variations in residual soils. However, most data points fall within the classification zone of silty sand to sandy silt, with a trend toward zone 12, indicating increasing relative density (D_r) and cemented silty sand. The presence of fines in this classification is consistent with grain size distribution analyses. Besides, three-dimensional Q_t:F_r:B_q projection [10] presents similar limitations, as most data points align along the B_q = 0 axis, albeit with less dispersion (Fig. 4). The evolution of qc with depth exhibits a consistent trend in the tests conducted at stations 6 and 7. Meanwhile, tests at station 3 and midway between stations 3 and 4 indicate higher resistance, consistent with the results obtained from the DPSH tests. The penetration tests reached a maximum depth of 3 to 3.2 m, primarily due to the high resistance of the soil and/or the reaction capacity limits of the equipment (100 and 200 kN). Granular soils tend to exhibit high qc values and low R_f. While sleeve friction remains relatively constant, localized increases are observed, which may be associated with differential alteration zones containing kaolinized porphyroid feldspar crystals and suction forces due to the partially saturated nature of these soils.

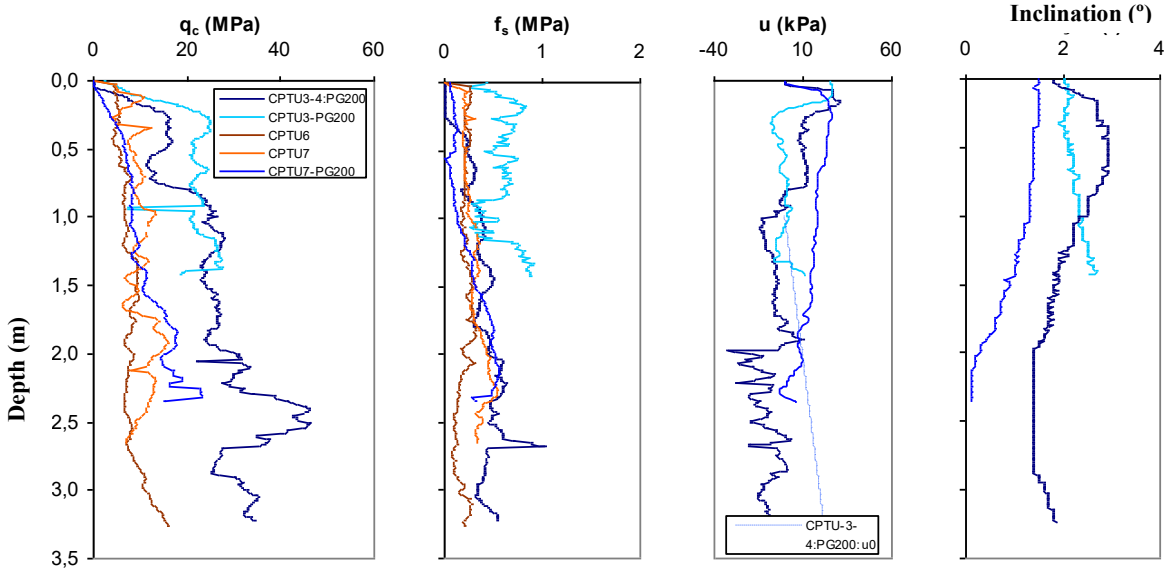
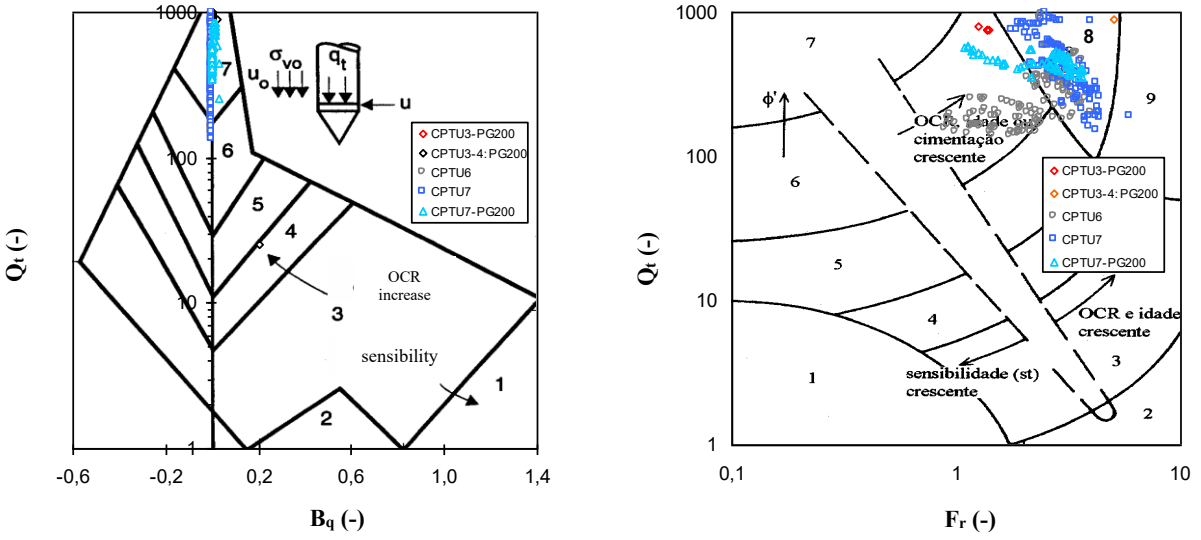


Fig. 3. CPTu results for q_c (a), f_s (b), u (c) and inclination (d)



1. sensible fine soil
2. organic (peat)
3. clays to silty clays
4. silty clays to clayey silts
5. sandy silts to silty sands
6. sands
7. sands to with gravel
8. dense sands to clayey sands (cemented or over consolidated)
9. hard fine soils (cemented or over consolidated)

Fig. 4. CPTU results *abacus (modified according to [10])*.

Despite these challenges, the q_t:σ'_{v0} projection [11] separates the test results into two distinct groups: station 3 and the midpoint between stations 3 and 4 representing a more resistant rock mass; and stations 6 and 7 where the rock mass shows greater weathering. The ϕ' values obtained from CPTu correlations appear overestimated, particularly in more resistant zones, compared to direct shear and triaxial laboratory tests. This discrepancy is attributed to soil structuring and interparticle bonding, which influence cohesion but are not accounted for in traditional correlations. Robertson (1990) [10] proposed a classification system that normalizes q_c/σ'_{v0} versus ϕ' and incorporates q_c as a function of modulus (M), elastic modulus (E), and small-strain shear modulus (G_0). These models assume similar mechanical behavior for soils with equivalent relative density and confining stress, which is not valid for residual soils. Advancements in penetration resistance theories for low-cohesion soils are hindered by soil heterogeneity and uncertainties in their properties and stress states.

Two SCPTu tests were performed at stations 7 and 3. Due to shallow groundwater levels, the focus was on V_s, as compressional wave velocity (P-waves) is strongly influenced by moisture content. The shear wave source consisted of a wooden beam generating impact-induced seismic pulses, initially placed in a distinct position. The tests were conducted with the seismic source positioned 1.5 m from the cone penetration point, with static vertical loading applied to the beam.

Two interpretation methods were applied to assess V_s, the reverse polarity technique, which uses two inverted-polarity seismic signals recorded at the same depth and compared with signals from subsequent depths, filtering is applied if necessary, and zero-crossing points are identified at the peak amplitude. Knowing the depth difference and time delay, V_s is determined. And the cross-correlation technique, which determines interval velocities using cross-correlation between seismic signals at different depths, the maximum correlation peak provides wave propagation time along the depth increment. The advantages of reverse polarity technique eliminate subjective arrival-time selection and use full seismic waveform rather than a single point, measuring signal similarity, ensuring higher reliability.

The non-linearity of stiffness with deformation is recognized as an important characteristic of earth materials, important as it constitutes a reference value that allows the classification of other deformability moduli, when defined for different deformation values. Based on the values of the shear wave propagation speed obtained from the SCPTu tests and the physical characteristics of the massif assessed using in situ sampling, namely the volumetric weight that was found to vary from the surface to the depth of investigation from 19.08-20.02 kN/m^3, the estimated values of the G_0 are presented in Table 2. The results of q_c, R_f, u, V_s and the estimated values of G_0, in the profiles obtained by the two tests are presented in Fig. 5.

Tab. 2. V_s and G_0 for two SCPTu tests.

Depth	**SCPTu7-PG200**			**Depth**	**SCPTu3-4-PG200**		
	Filter (Hz)	**Vs (m/s)**	**G_0 (MPa)**		**Filter (Hz)**	**Vs (m/s)**	**G_0 (MPa)**
1.5m	50-300	585.25	715	1.25m	20-500	604.69	715
2.45m	50-300	264.51	230	2.25m	20-500	338.69	229
				2.975m	20-500	315.07	202

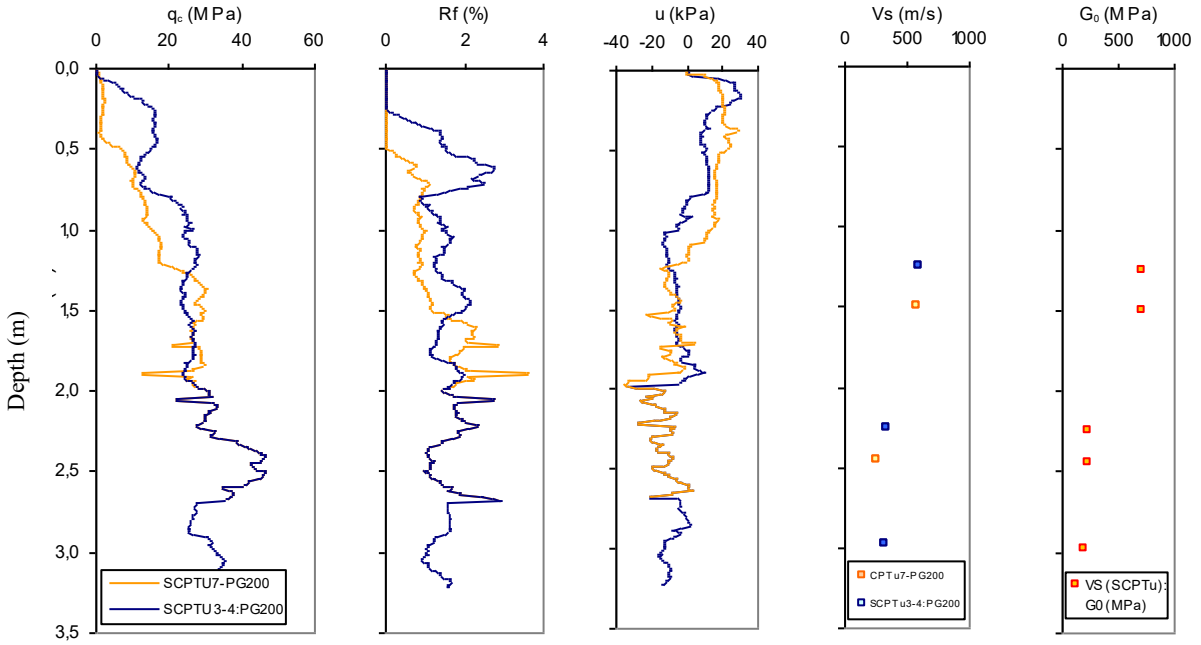


Fig. 5. SCPTu results for q_c (a), R_f (b), u (c), V_s (d) and G_0 (e).

4. Discussion

Residual soils generally exhibit partial saturation, and above the water table, suction mechanisms develop, leading to low B_q values close to zero. Under these conditions, resistance increases due to surface tensile effects caused by negative pore water pressure. However, capillary effects are difficult to sustain in large dynamic spaces, potentially existing between the soil and the cone tip [12]. The transition across the water table (NF at approximately 1.0 m depth from the embankment surface) results in a decrease in u2, sometimes reaching negative values due to the dilatant behaviour of these soils. Below the cone tip, the soil generally attains a critical density corresponding to the applied stress state. For shallow depths, plasticity manifests under low

stress conditions, and the critical density is lower than the initial soil density, leading to soil dilation. At greater depths, the mean stress at the time of penetration is higher than the initial density, causing compression and volume reduction. Consequently, soil beneath the cone tip compresses relative to the overlying soil, resulting in a decrease in vertical stresses in that plane.

As the penetration progresses, the cone continuously encounters new soil layers, and the process repeats, demonstrating that qc is directly related to the mean stress in the plasticity zone. Applying vertical stress correction to q_c and f_s reduces data dispersion, concentrating the points within the domain of over consolidated or cemented soils, which exhibit increasing ϕ'. This trend aligns with laboratory classification results. Interpreting piezocone test results in unsaturated soils require careful consideration, as Q_t and B_q factors are influenced by OCR and soil suction conditions. Accurate determination of shear strength parameters relies on u2 measurements, especially below the water table [13].

For cemented soils, the plasticity plane inclination relative to the horizontal is negative, as these soils tend to dilate underloading. This behaviour corresponds to dilatancy angles observed in triaxial tests on intact samples. The nonlinear relationship between q_c and σ'_{v0} reflects dilatant and peak strength characteristics [14]. Comparative studies between calibration chamber tests and theoretical models in sands with varying density and cement content confirm that qc values correlate with peak strength parameters, which is consistent with the brittle behaviour of cemented soils [15, 16]. A study [11] proposed a linear relationship between qc and σ'_{v0} for a given constant ϕ', which is valid for uncemented granular materials. However, in residual soils, this assumption is not entirely applicable due to their complex microstructure.

Using CPTu tests on the same granitic residual soil [17, 18], empirical correlations were applied to estimate shear strength parameters and relative density. The values showed less conservative than those obtained from direct shear and triaxial tests on structured samples, being direct shear box results: c' = 10 kPa, ϕ' = 29° (h = 3 m); c' = 20 kPa, ϕ' = 30° (h = 1.4 m); triaxial test: c' = 7.6 kPa, ϕ' = 29.6° for specimens from 1.4–3.5 m depth. Discrepancies between laboratory and in situ test results are primarily due to cemented soil structure inherited from the parent rock, which affects cohesion but is not accounted for in conventional correlations.

5. Conclusion

In the selected rock mass, an *in situ* testing program was previously designed to characterize it geotechnically. Both indirect characterization tests (electrical resistivity, seismic refraction, and seismic cone tests) and direct mechanical characterization tests (dynamic and static penetration) were conducted. The results revealed that the stations with the highest penetration resistance (N20 > 70) were between 3 m and 4 m deep, with increasing depth in stations 6 and 7 (h > 11 m). The tests also highlighted a significant reduction in alteration with depth, leading to an increase in soil rigidity. A comparative analysis between DPM and DPSH results showed good agreement up to depths of 3 m to 4 m, beyond which axial misalignment effects started to influence the results. The SCPTu tests indicated that below the water table, pore pressure exhibited negative variations due to the dilatant behaviour of the residual soil. The CPTu and SCPTu classifications aligned with the penetration resistance trends observed in the dynamic tests. The SCPTu results confirmed that the studied soils predominantly fall within silty sand and dense sand categories, with increasing cementation tendencies at greater depths. Nevertheless, the soils were classified as sand to silty sand, with high and increasing OCR and D_r values, transitioning towards a cemented soil. Below the water table, pore pressure exhibits negative variations due to the dilatant nature of these soils. The evaluation of the ϕ' using classical correlations yields high values, which is likely due to the inability to separate the effects of friction, dilatancy, and a possible weak cohesion related to residual bonding forces between minerals.

The combination of SCPTu, CPTu, and dynamic penetration tests provides a comprehensive dataset for geotechnical assessment, reducing uncertainties in subsurface investigations, ensuring greater reliability for foundation design and infrastructure projects. Future studies should explore additional correlations between in situ and laboratory tests to further refine soil classification methodologies in residual and cemented soils.

Acknowledgments

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