Deriving strength parameters of granitic residual soils from Ménard pressuremeter tests

Carlos Rodrigues[1], *Carin* Mateus[2] and *Nuno* Cruz[2#]

[1]*Polytechnic Institute of Guarda, Civil Eng. Dept., Avenida Dr. Francisco Sá Carneiro, 50, 6300-559 Guarda, Portugal*
[2]*Mota-Engil Global, Geology and Geotechnics, Rua Rêgo Lameiro, 38, 4300-454 Porto*
[#]*Corresponding author: nunocruz@mota-engil.pt*

ABSTRACT

Residual soils are non-textbook materials that are hard to be modelled by traditional soil mechanics, which creates serious difficulties in the in-situ test interpretations and the consequent applications to geotechnical design. This is due to the presence of a cementation structure that is responsible for a cohesive-frictional behaviour of these soils, meaning that two strength parameters must be derived to represent the overall strength. Furthermore, cementation structure also deeply affects the stiffness behaviour, deviating from typical response of transported soils. The common interpretation of in-situ tests usually considers extreme behaviours represented by only one parameter, namely undrained cohesion for clays and angle of shearing resistance for sands, which naturally do not work in these cohesive-frictional materials. For this purpose, only tests that take two or more independent measurements can be used to solve this problem, as it is the case of SCPTu, SDMT and PMT tests, while SPT and DPSH tests cannot be effective in this determination.
Portuguese research institutions have been looking over the granitic residual soil characterization through specific research works, from which resulted several publications on the subject. Following previous research works of calibration with (S)DMT (Cruz, 2010) and (S)CPTu (Cruz et al. 2018) tests performed in Polytechnic Institute of Guarda (IPG), Portugal, a new research frame was developed to settle a methodology for obtaining strength parameters of granitic residual soils from pressuremeter tests (PMT), which is presented and discussed herein.

Keywords: Residual soils; cemented structure; shear resistance; triaxial test; PMT.

1. Introduction

Aiming to study the behaviour of residual soils and their deviations from the typical patterns observed in sedimentary soils a global research program has been undergoing in Polytechnic Institute of Guarda (IPG) based in several in-situ and laboratory tests performed on a restricted experimental site that includes the local residual granitic massif.

The studied area (Guarda, northeast of Portugal) is warm-summer, temperate, Mediterranean climate (Csb-Koppen climate classification), which favours the weathering of the granitic rock mass that dominates the region, turning the granite masses into a permeable sandy frame. The fluctuations of water level that go from a submerged stage in the wet season followed by drying up to depths of 5 to 6 meters during the summer, create the conditions to favour the constant weathering of the rocky substrate. As weathering progresses, the primary interparticle bonds between the grains are broken and a series of intergranular voids are created. Afterwards, weathering makes the feldspars and micas unstable, allowing leaching to occur, with the creation of a network of intragranular voids. In addition, the more stable minerals, mostly quartz grains, are bonded by highly weathered (and therefore unstable) grains of feldspars and micas to form a solid skeleton that can be quite open.

In the last 20 years several research frames were established to understand the general behaviour of these soils and analyse modes of interpretation specifically dedicated to these soils.

The last testing program consisted of 6 SDMT tests, 6 SCPTu tests and 18 PMT tests distributed by 6 boreholes, as well as 3 triaxial tests with internal instrumentation, performed in undisturbed and reconstituted samples, as represented in Fig. 1. In this framework, emphasis is given to PMT and triaxial tests carried out on selected samples as discussed hereafter.

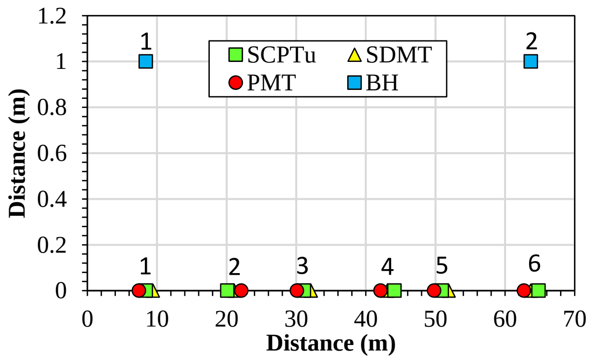

Figure 1. Spatial localization of the PMT tests and boreholes (BH)

One of the main goals of these frames was to establish specific models to obtain simultaneously cohesive intercept and angles of shearing resistance of these cohesive-frictional materials, which is not covered by the common interpretation models used in sedimentary soils.

At the time of carrying out the field tests, the water level was located at the depth of: point 1, 1.25 m; point 2, 1.40 m, points 3, 4 and 5, 1.50 m; point 6, 1.70 m.

Fig. 2 shows some characteristics of the soil deposit in analysis in this work.

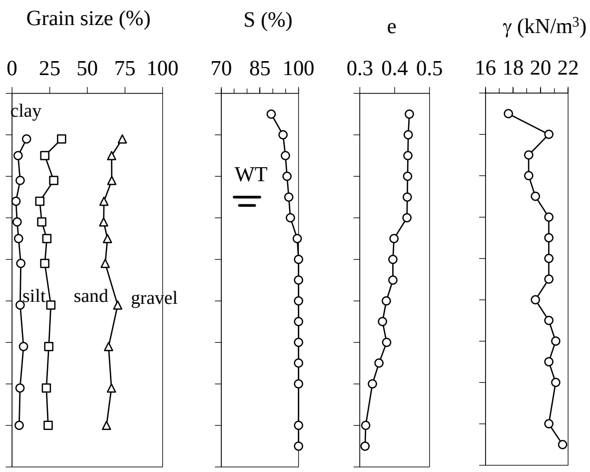

Figure 2. Soil characteristics of the massif under study; S=degree of saturation; γ=unit weight; e=void ratio

2. Residual soil behaviour

Saprolites are materials that result from the weathering of an original unweathered rock massif. Evolution of mechanical behaviour with weathering is rather complex to follow and depend on many variables, such as the geologic nature, presence of minerals of strong influence, anisotropy structures, among others. The subject represents a particular domain of geotechnical engineering that has been widely studied and published.

With advancing weathering mechanical parameters decrease, showing a tendency for the cohesive intercept (in terms of Mohr-Coulomb failure envelope) to be reduced by relaxation of grain boundaries and micro-fracturing, while angles of shear resistance are slightly higher than the same soil in a remoulded state. Globally, the loss of strength with weathering can be fairly represented by a reducing cohesion intercept (c’) due to weakening of contact forces. The cohesive intercept is present even when soils show strong contraction during shear and can be a result of many other contributions apart from bonding, such as electrostatic forces, adhesion of fine particles, contact cementation developed with time and pressure (ageing), and suction due to development of negative pore pressures in unsaturated conditions. In the most part of situations chemical bonding and suction give the fundamental contribution for strength (Viana da Fonseca & Coutinho, 2008). The loss of strength is naturally followed by an increasing deformability that results from the increasing porosity and from the decreasing cementation magnitude.

Globally, at any stage, saprolites are characterized by variable grain strength as function of mineralogy, the presence of a bonding structure that influences strength and stiffness and a void ratio highly influenced by the weathering level. The continuous evolution of both the grain size distribution and the variable density along weathering, leave no space for stress history. These characteristics contrasts with those observed in sedimentary soils where stress history play an important role in strength and stiffness behaviour, the grain strength is uniform (weaker particles are eliminated during transport phases), the void ratio depends directly on stress history and cemented structures only occurs in geologic aged deposits (Brenner et al., 1997).

3. Characterization of saprolites

Strength characterization of saprolites face some challenges, since the current interpretation models dedicated to transported soils cannot represent adequately the residual masses. In the context of strength, the actual in-situ tests interpretation methodologies do not allow to derive the two strength parameters. In fact, the common interpretation of in-situ tests (sedimentary soils) usually considers extreme behaviours represented by only one parameter, namely undrained cohesion for clays and angle of shearing resistance for sands, which naturally do not work in these cohesive-frictional materials. The main consequence of this in residual soils is that the derived angle of shearing resistance is overestimated because it integrates both cohesion and the real friction. Therefore, to have a correct in-situ strength characterization of saprolites new methodologies are required, which must guarantee the simultaneous calculation of the cohesive intercept and the angle of shearing resistance. Naturally, for this purpose, only multi-parameter tests can be used to solve the problem, as it is the case of (S)CPTu, (S)DMT and PMT tests, while SPT and DPSH tests cannot be effective in this determination. In the case of (S)DMT and (S)CPTu tests the approach was to establish a correlation to obtain the cohesive intercept and then correct the friction angle derived from sedimentary approach as function of the magnitude of cohesive intercept (Cruz 2010, Cruz et al. 2018). Finally, it should be underlined that cementation also affects the stress-strain behaviour, thus the sedimentary approaches cannot represent with accuracy the response of residual soils. However, this subject is out of scope of the discussion presented herein.

Triaxial tests allow for the determination of the two parameters, but they are not efficient to cover the usual heterogeneity of weathered masses, because the number of tests possible to integrate in current campaigns is generally insufficient to do so. Furthermore, tests depend on the quality of samples, but sampling affects the cementation structure, compromising the results of the cohesion intercept. As so, in-situ strength characterization is fundamental for the correct strength characterization of materials arising from weathering, requiring continuous profiling spaced according to the perceived local heterogeneity of the residual mass.

Previous research on the subject allowed to obtain specific methodologies for deriving both parameters from (S)DMT (Cruz 2010, Cruz et al. 2014) and (S)CPTu tests (Cruz et al. 2018). The earlier were obtained by means of a calibration experience working with artificially cemented granitic mixtures tested in an IPG calibration apparatus supported by parallel triaxial tests, which allowed to avoid the sampling disturbance (Cruz 2010). CPTu correlations were obtained by calibration from DMT test results (Cruz et al 2018). Original data was re-analysed in 2022, after the completion of another research frame. These methodologies developed in IPG granites were then tested in Porto granites where several experimental sites with parallel DMT and CPTu tests are available, supported by important data represented in Porto Geotechnical map (COBA, 2003) that allowed to establish geotechnical evolution through weathering (Cruz 2010, Cruz et al 2015). The obtained correlations are represented by Eq. 1 and Eq. 2 related with DMT tests and Eq. 3 and Eq. 4 related with CPTu.

$$c'_g = 7.716 \ln(vOCR) + 2.94 \quad (1)$$

$$\emptyset_{corr} = \emptyset_{sed} - 3.45 \ln(vOCR) + 8.20 \quad (2)$$

$$c'_g = 11.5\, ln(Q_{t1}) + 3.2\, ln(F_R) - 30.8 \quad (3)$$

$$\emptyset_{corr} = \emptyset_{sed} - 5.27\, ln(Q_{t1}) - 0.99\, ln(F_R) + 22.46 \quad (4)$$

where c'_g is the global cohesion generated by the cementation and suction; Q_{t1} and F_R are CPTu intermediate parameters (Robertson and Cabal 2015); vOCR is the virtual overconsolidation derived from Marchetti & Crapps (1981); ϕ_{corr} is the corrected angle of shear resistance; ϕ_{sed} is angle of shear resistance derived from sedimentary correlations based in DMT (Marchetti 1997) and CPTu tests (Robertson and Cabal, 2015).

4. PMT interpretation

Deriving strength parameters from PMT tests in sedimentary soils follows the same approach of DMT and CPTu tests, where an angle of shearing resistance is derived in granular soils, while in cohesive soils the undrained cohesion is obtained. In the case of granitic saprolites the correspondent soils are clearly granular, thus the angle of shearing resistance is the resulting parameter. In such case, the existent correlations for deriving the parameter are the proposals of Ménard (in Baud, 2020) and Hughes et al. (1977).

Ménard Pressuremeter test is based in a different methodology when compared with DMT and CPTu tests. The main difference is that in PMT tests a stress-strain curve is generated by applying several load increments measuring the correspondent volume variation, while DMT obtain only two different pressures related with two displacements of the membrane moving outwards (0.05 mm and 1.10 mm). Both are supported by principles of Theory of Elasticity, namely the expansion of cylindrical and semi-spherical cavities, respectively related with PMT and DMT tests. In the case of CPTu, at any specific depth, a triple reading is obtained, namely point resistance, side friction and pore pressure.

The PMT field measurements allow to create a pressuremeter curve, from which the lift-off pressure (P_0), creep pressure (P_f), limit pressure (P_{lM}) and pressuremeter module (E_M) are obtained. The important thing for the present study is that the P_f and P_{lM} represent the strength at different strain levels, respectively at the end of pseudo-elastic phase and at the vicinity of failure, respectively. On the other hand, there are two main correlations for deriving angles of shearing resistance, one depending on P_{lM} (Ménard, *in* Baud, 2020) and another depending on the pressuremeter curve (Hughes et al. 1977), which will generate results related with different strength states of the tested soil. As consequence of these aspects, two approaches for obtaining representative correlations for residual soils can be settled:

1) Derive the correct angle of shearing resistance from Ménard correlation, which correspond to a stress-strain *locus* where the cementation structure is almost completely destroyed; derive the angle of shearing resistance from Hughes et al. proposal where the cementation is present, which will be higher than Ménard's because it integrates friction and cohesion in the same parameter; the resulting difference from the two approaches can be worked using Mohr-Coulomb envelope two obtain the correspondent cohesion
2) Use the combined characteristic pressures, P_f and P_{lM}, to obtain correlations with cohesion and angles of shearing resistance.

The two approaches were followed giving interesting results, but for space reasons, only the first approach is presented and discussed herein, leaving the second one to a forthcoming publication.

4.1. Triaxial testing

To learn about the shear strength characteristics of the local granitic residual soils a set of isotropically consolidated drained (CID) triaxial tests was performed on samples collected at depths ranging from 0.50 m e 3.40 m. The shear phase was executed at rates of 0.015%/min with drainage by the two ends of the sample, aiming to ensure the complete dissipation of pore water pressure (Rodrigues 2003, Cruz 2010). Axial strains were measured by means of an external transducer of high resolution and submerged internal transducers of LVDT type. Deviatoric stresses was obtained by means of a submerged cell of 10 kN.

Three groups of triaxial tests were executed: one group related with samples retrieved in BH1 at depths within 0.50 m and 1.50 m, a second one on samples obtained in BH2 at depths of 0.50 m and 1.50 m and a third one at depths of 1.50 to 3.50m of BH2.

4.1.1. Strength envelopes

The envelopes related with strength peaks (q_{max}) and critical state (q_{cs}) plotted in q-p' space [$q=\sigma'_1-\sigma'_3$ and $p'=1/3(\sigma'_1+2\sigma'_3)$] are presented in Fig. 2 to Fig. 4, revealing the linearity of the de-structured materials and the non-linearity of the structured ones, which increases with the cementation level.

The results show a strong resistance of the strongly cemented soils, reflected by a significant cohesive intercept, with a highly dilatant behaviour and a tensile strength arising from the cohesive state. The dilatant component arises from the low void ratio and the imbricated micro-fabric inherited from the parent rock.

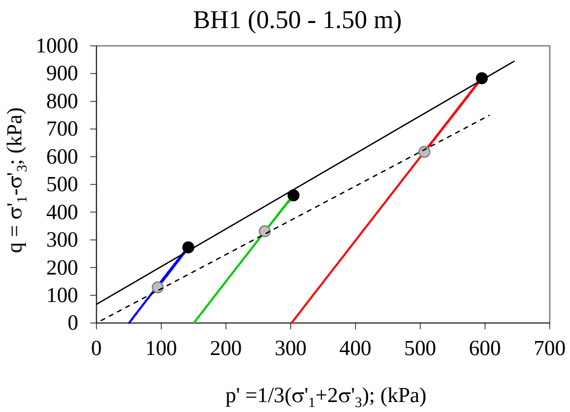

Figure 2. Failure envelopes of the q_{max} and critical state, of the materials collected in the BH1 (0.50-1.50 m).

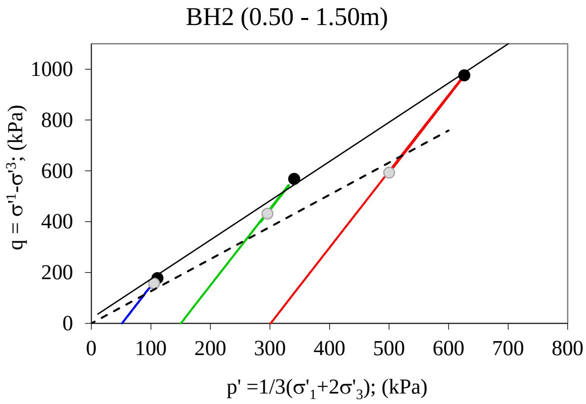

Figure 3. Failure envelopes of the q_{max} and critical state, of the materials collected in the BH2 (0.50-1.50 m).

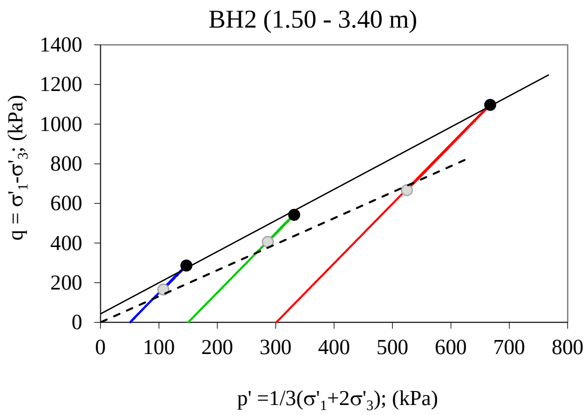

Figure 4. Failure envelopes of the q_{max} and critical state, of the materials collected in the BH2 (1.50-3.40 m).

4.1.2. Shear resistance parameters

A summary of the shear strength parameters obtained in the set of triaxial tests is presented in Table 1, revealing a cohesive parameter ranging from 10 to 33 kPa, while peak and critical angles of shearing resistance vary from 34º to 38º and 31º to 33º, respectively.

Table 1. Peak and critical shear strength parameters

Sample	q_{max}		critical state
	c' (kPa)	ϕ' (º)	ϕ'_{cs} (º)
BH1 (0.50 – 1.50 m)	33.1	33.7	30.8
BH2 (0.50 – 1.50 m)	10.0	37.8	31.5
BH2 (1.50 – 3.40m)	21.6	38.5	32.6

4.2. Ménard pressuremeter tests (PMT)

4.2.1. Obtained results

Eighteen PMT tests were performed in the field within 6 boreholes, as previously presented in Fig. 1, at depths of 0.50, 3.00 and 4.50 m. The tests were performed according to the Ménard recommendations (1975) and following the standards ISO 22476-4 (2021) and ASTM D 4719 (2020). Since this paper lies on the comparison of PMT and triaxial test results, only PMT 1 and PMT 6 groups are presented (Fig. 5 and Fig.6). The pressuremeter curves show clearly the 3 load phases: recompression, pseudo-elastic and plastic. The lift-off pressures (P_0), creep pressures (P_f), limit pressures (P_{lM}) and pressuremeter module (E_M) are presented in Table 2.

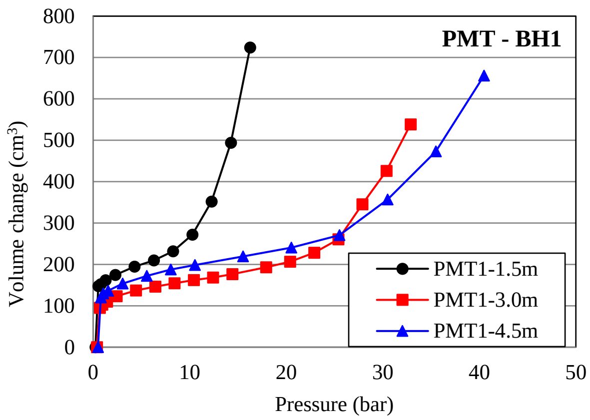

Figure 5. Corrected pressuremeter curves of PMT1 group.

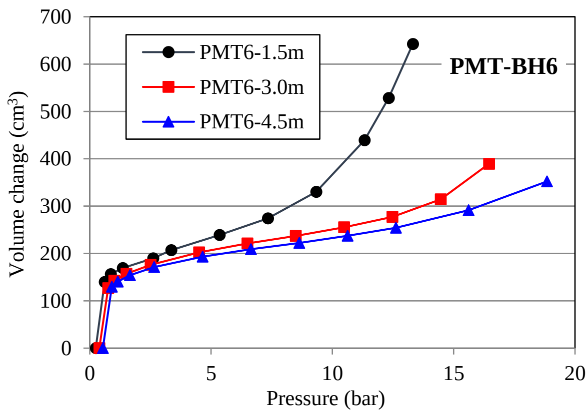

Figure 6. Corrected pressuremeter curves of PMT6 group.

Table 2. PMT test results

Test	Depth (m)	P_0 (MPa)	P_f (MPa)	P_{lM} (MPa)	E_M (MPa)
PMT1.1	1.5	0.13	1.07	1.64	19.27
PMT1.2	3.0	0.24	2.13	3.48	40.67
PMT1.3	4.5	0.16	2.73	4.14	37.92
PMT6.1	1.5	0.26	0.90	1.42	10.83
PMT6.2	3.0	0.15	1.20	2.13	20.85
PMT6.3	4.5	0.17	1.43	2.60	23.76

4.2.2. Evaluation of shear resistance parameters by the PMT tests

The angle of shearing resistance was firstly obtained by the Ménard (*in* Baud, 2020) proposal that is presented below (Eq. 5):

$$p_{lM} = k.2^{(\emptyset'-24)/4} \quad (5)$$

k is 2 for wet sand, 3 for dry sand and 2.5 on average.

The PMT results derived this way that are comparable with triaxial results are presented in Table 3, considering k=2.5, which are in the neighbourhood of the peak triaxial results. Note that the Ménard correlation is based in the limit pressure (P_{lM}) and the implicit strain is quite high with the soil in the vicinity of failure. In the residual soils, in such state the cementation structure is no longer present, thus the strength is purely frictional.

Table 3. Angles of shearing resistance (ϕ') derived from Ménard proposal

Test	Depth (m)	ϕ' (º)
PMT1.1	1.5	34.81
PMT6.1	1.5	33.89
PMT6.2	3.0	36.42

On the other hand, Hughes et al. (1977) provides a method for the determination of the peak angle of shearing resistance for dense sands using the procedure illustrated in Figure 7.

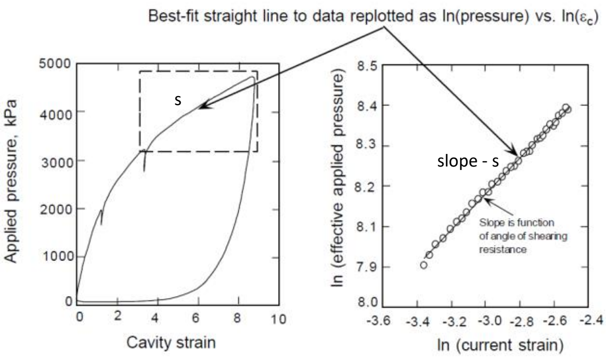

Figure 7. Determination of angle of shearing resistance from SBPM tests in sand (After Clarke, 1997)

Initially the slope of the latter part, *s*, of the curve is determined and the value of the angle of shearing resistance at constant volume, ϕ'_{cv}, is selected from the Table 4 for the respective soil type.

Table 4. Typical values of ϕ'_{cv} (after Robertson and Hughes, 1986)

Soil type	ϕ'_{cv} (º)
Well-graded gravel-sand-silt	40
Uniform coarse sand	37
Well-graded medium sand	37
Uniform medium sand	34
Well-graded fine sand	34
Uniform fine sand	30

Then the angle of shearing resistance is evaluated using the relationship given in Eq. 6. As ϕ'_{cv} is not critical in determining ϕ', Clarke (1996) recommends a value of 35° to be used.

$$\sin\phi' = \frac{s}{1+(s-1)\sin\phi'_{cv}} \quad (6)$$

Angle of dilation can also be determined using the relation given in Eq.7 using the values of *s* and ϕ'_{cv}.

$$\sin\psi = s + (s-1)\sin\phi'_{cv} \quad (7)$$

The method of Hughes et al. (1977), developed for the SBPM tests, was suitable for the PMT test, and in the case of the tests carried out, presented the results shown in Fig. 8 and in Table 5.

Table 5. Angles of shearing resistance (ϕ') obtained by the proposal of Hughes et al., (1977)

Test	Depth (m)	s	ϕ'(º)
PMT1.1	1.5	0.626	52.74
PMT6.1	1.5	0.604	50.82
PMT6.2	3.0	0.524	46.92

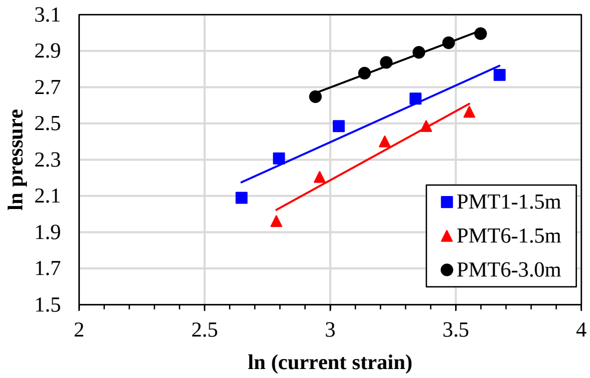

Figure 8. The determination of angle of shearing resistance from PMT tests (adapted method of Hughes et al. 1977)

The results obtained clearly illustrate, compared with the results obtained from triaxial tests, that the value of the shear resistance angle is significantly overestimated. One of the explanations for these results is related to the fact that pressure-volume records lower than P_{lM} contribute to the evaluation of parameter s, which should also mobilize a substantial cohesive portion of the shear resistance.

5. Method to estimate c' and ϕ' from PMT tests

In this work, a method was developed to estimate the value of the shear resistance parameters (c', ϕ'), in the soils under analysis, using the results of the PMT tests, and the values of ϕ' determined by the proposals of Ménard (Baud, 2020) and Hughes et al. (1977). The method consists of the following (Figure 9):

a) The value of ϕ' estimated by Ménard's proposal is assumed to be the peak of shear resistance angle, ϕ'_p;
b) The value of ϕ' evaluated by the proposal by Hughes et al. does not correspond to the value of ϕ'_p, as it appears overestimated. However, this value reflects the shear resistance mobilized for an effective stress, corresponding to the depth at which the PMT test was carried out;
c) The failure envelopes corresponding to the values of ϕ' obtained by the proposals of Ménard

and Hughes et al. are then plotted into the stress space τ-σ_N;

d) The in-situ effective stress (σ'_{v0}) is determined at the point where the PMT test was carried out;

e) In the space of stresses τ-σ_N, a vertical straight line is passed through $\sigma'_{v0,}$ which is considered the value of normal stress σ_N, which intersects the failure envelope corresponding to ϕ' determined by the proposal of Hughes et al.;

f) Through the intersection point, a straight line is passed with the slope of ϕ' determined by Ménard's proposal;

g) The straight line corresponds to the failure envelope of the residual granite soil, making it possible to know the value of the cohesive portion of the resistance, which corresponds to the intersection at the origin of the straight line.

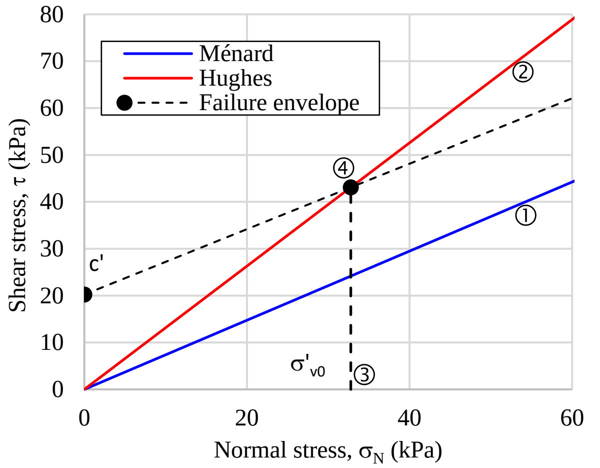

Figure 9. Proposal to estimate the real shear resistance parameters (c', ϕ') from PMT tests

The application of the previously method results in values for the shear resistance parameters (c', ϕ'), for the granitic residual soils under study, which are presented in Table 6.

Table 6. Shear resistance parameters obtained by through the combination of proposals of Ménard (in, Baud, 2020) and Hughes et al. (1977)

Test	Depth (m)	c' (kPa)	ϕ' (º)
PMT1.1	1.5	19.1	34.8
PMT6.1	1.5	15.9	33.9
PMT6.2	3.0	20.5	36.4

The results presented in the Table 6, when compared with those from triaxial tests, carried out on samples collected from adjacent holes and at the same depths, presented in Table 1, allow us to observe that the order of magnitude of the parameters is similar. It is also observed that the value of the shear resistance parameters, obtained by the present method, resulting from the PMT tests, is slightly lower than that determined by the triaxial tests. It should be noted that if the sampling problem causes a reduction in the real value of the shear resistance parameters, the opening of the borehole to installing the pressuremeter probe may disturb the massif more than the sampling. It should be noted that the samplers used to collect the samples to carry out the triaxial tests were thin-walled samplers, with 70 mm internal diameter, with an area index of 18%, a ratio L/D=4.7 and B/t=27.1. Furthermore, the samplers were fixed statically, which made it possible to significantly reduce the effect of sampling disturbance (Rodrigues and Lemos, 2001).

In a similar way, we can compare the results of the shear resistance parameters obtained by the PMT tests with those obtained by the formulations for the DMT (Eq. 1 and Eq. 2) and CPTu (Eq. 3 and Eq. 4) tests. Figure 10 and Figure 11 represent the basic parameter profiles obtained in CPTu and DMT tests, respectively, while in Figure 12 and Figure 13 cohesion (c') and angles of shearing resistance (ϕ') obtained by DMT, CPTu and PMT tests are represented together for direct comparisons.

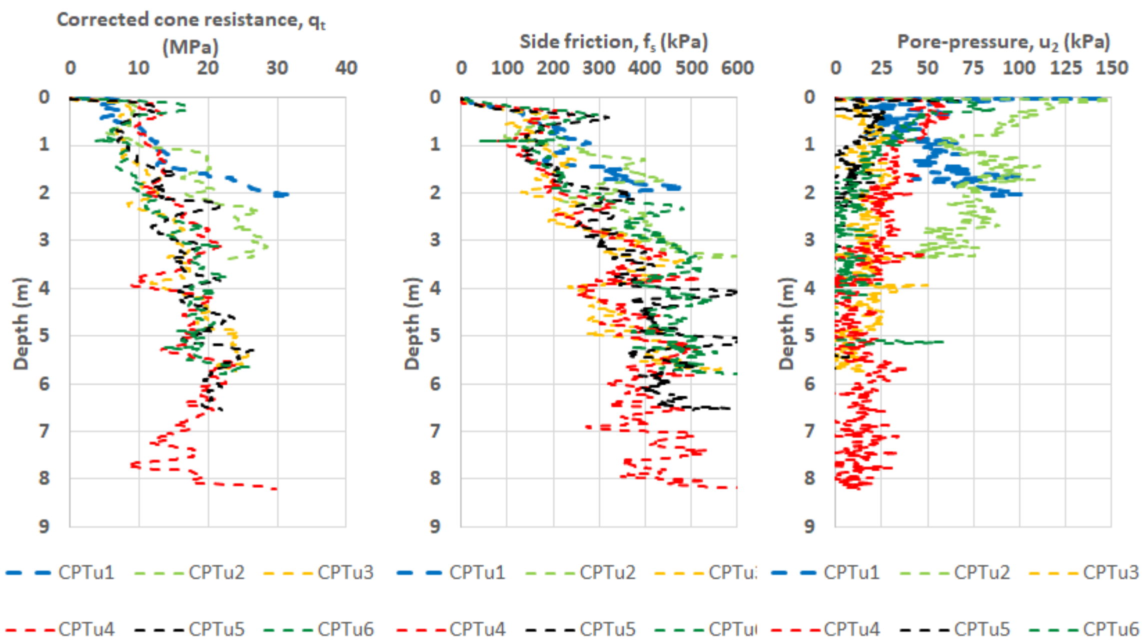

Figure 10. CPTu basic parameter profiles

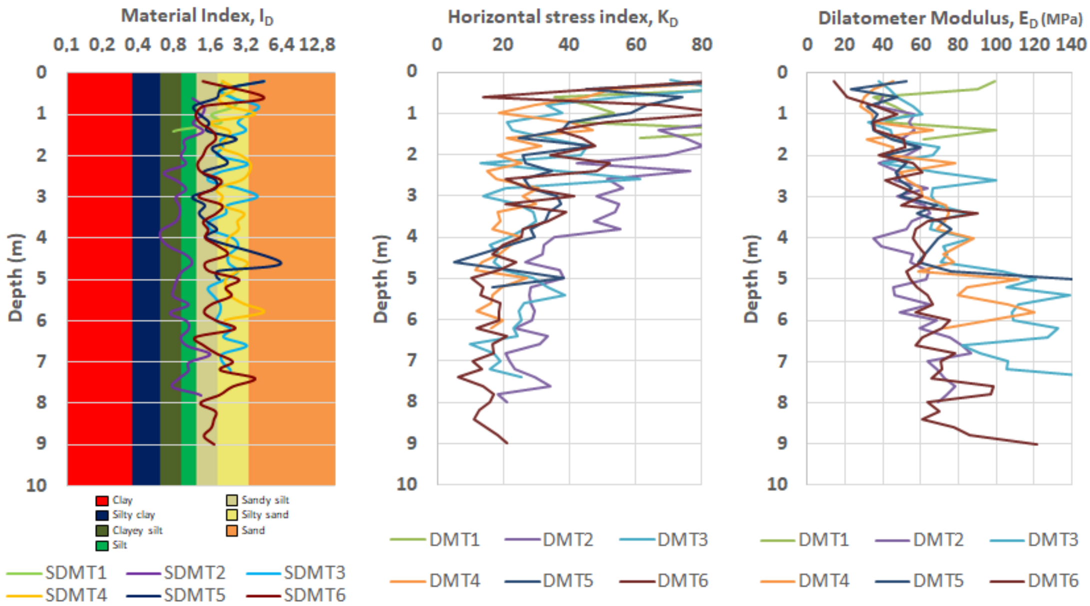

Figure 11. DMT basic parameter profiles

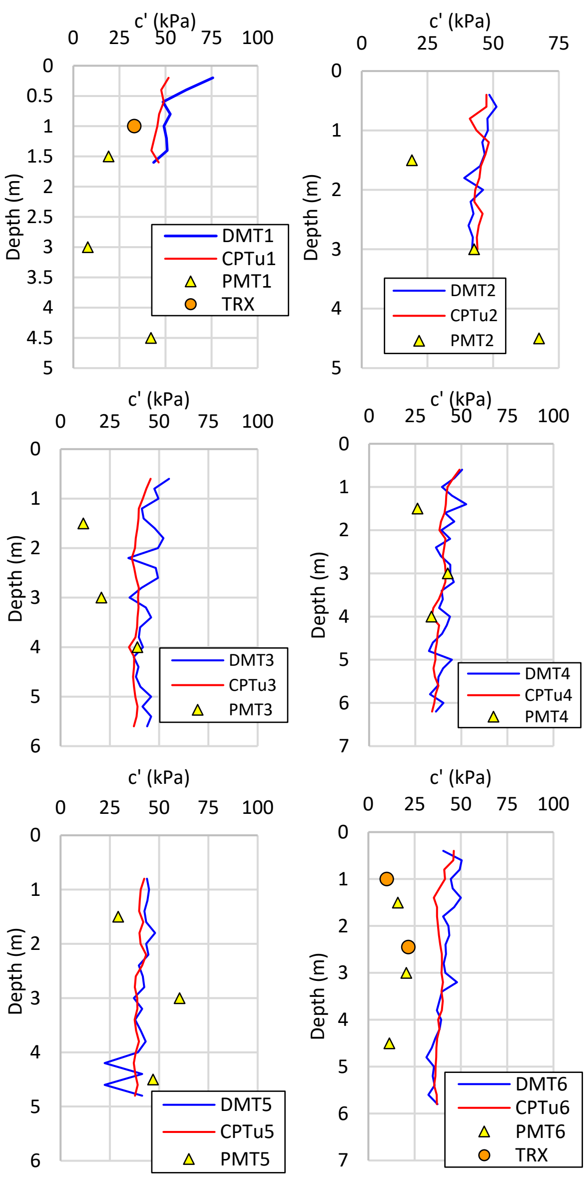

Figure 12. Cohesion evaluated from PMT, DMT and CPTu

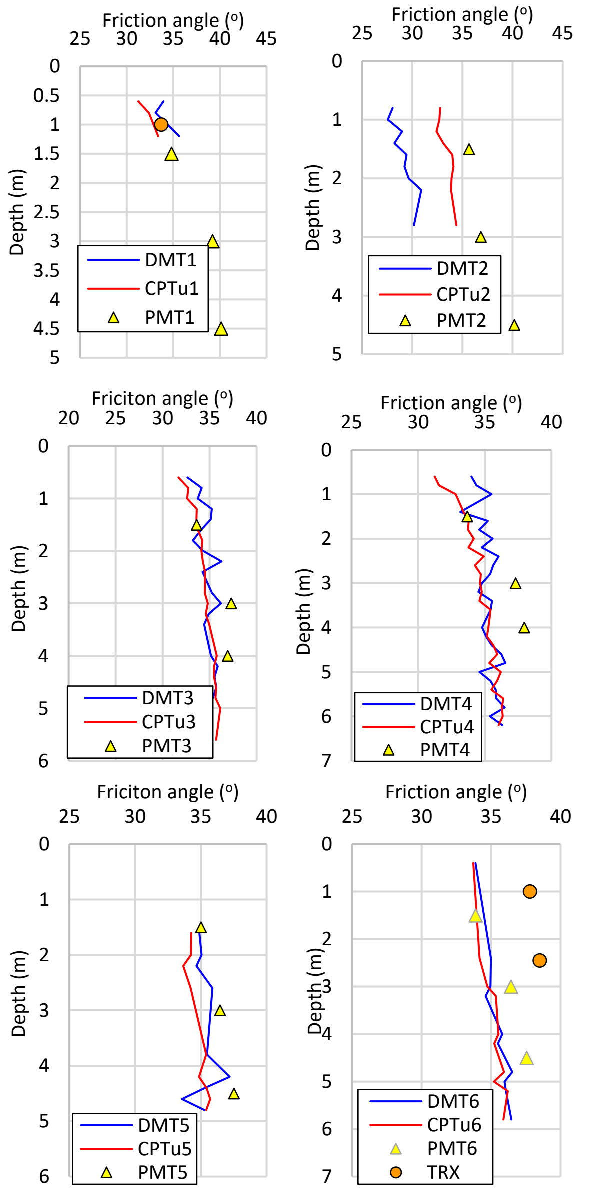

Figure 13. Friction angle evaluated from PMT, DMT and CPTu

The results obtained allow us to verify that the various formulations used to estimate the value of the cohesive parcel of the resistance (c'), by the in-situ tests (CPTu, DMT and PMT), are of the same order of magnitude. However, the results obtained seem to indicate that the values produced by the PMT tests are slightly lower and generate a greater dispersion.

The comparison of the results of the parameter ϕ' is shown in Figure 13. Like c', the results obtained by the various methods show that the order of magnitude of the estimated value of ϕ' is of the same magnitude. However, the dispersion, in this case, is smaller.

As can be seen in Figures 12 and 13, the values of the shear resistance parameters (c and ϕ) estimated by in-situ tests (PMT, CPTu and DMT) are of the same order of magnitude as those evaluated by triaxial tests. It should be emphasized that at DMT6/CPT6/PMT6 location, there is a gap between the global results of in-situ tests and triaxial, showing higher angles of shearing resistance somehow compensated by the lower deduced cohesion intercept, which might probably related with singularities present in the location of the collected samples.

6. Conclusions

The application of Ménard's proposal (in Baud, 2020) to PMT tests, carried out on granitic residual soils, to estimate the value of the shear resistance angle (ϕ'), produces values very close to those obtained by triaxial tests. However, the proposal does not allow estimating the value of the cohesive parcel (c'). The application of the proposal of Hughes et al. (1977), using the same results from the PMT tests, allowed us to verify that the value of ϕ' is overestimated. The explanation for these results must be related to the level of extension to which the parameters involved in the proposals used (Ménard and Hughes et al.) drive the materials. P_{lM} in the case of Ménard and P_f to P_{lM} in the case of the proposal by Hughes et al.

In this work, a proposal was developed to estimate the value of the shear resistance parameters (c' and ϕ') of the granitic residual soils under investigation, which combines the proposals of Ménard and Hughes et al. The results obtained allow us to verify that the results of c' and ϕ' estimated by the proposal now presented are close to those obtained in triaxial tests. It was also found that when we compare the values of c´ and ϕ´ estimated by the proposal based on the PMT tests, with other formulations developed for the CPTu and DMT tests, the value of c´ estimated by the proposal based on the PMT tests generates slightly lower values and with greater dispersion than those obtained by the CPTu and DMT tests. The parameter ϕ' estimated by the various proposals presents low dispersion and very similar values.

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