An Approach to Derive Strength Parameters of Residual Soils from DMT Results

N. Cruz, C. Rodrigues, A. Viana da Fonseca

Abstract. Residual soils show a specific mechanical behaviour classified as non-conventional when compared with sedimentary transported soils, since the presence of a cemented matrix plays an important role on their strength and stiffness. Moreover, suction is frequent in natural profiles, which in residual soils creates several problems on the interpretation of in-situ test results. These two factors, cementation and suction, are contributing simultaneously as structuring factors. Correlations to deduce strength parameters in Portuguese granitic residual soils by Marchetti Dilatometer were previously established under a data base obtained in careful triaxial testing programs executed on “undisturbed” samples. However, the reference results were affected by sampling disturbance and space variability, and therefore somehow deviated from in-situ conditions. To solve these problems, a large calibration box was constructed to work with artificially cemented soils where DMT blades could be pre-installed and pushed-in. Water level, suction and seismic velocities were monitored during the experiment and a triaxial program was established in parallel on the same artificially cemented mixtures. As a result, specific correlations to derive the cohesion intercept value and the angle of shearing resistance in saturated and unsaturated conditions were developed and subsequently tested in a well characterized experimental site. Herein, the results of that experimental framework are presented and discussed.

Keywords: residual soils, Portuguese granitic formations, cohesion intercept, angle of shearing resistance, suction, in-situ characterization, DMT.

1. Introduction

Residual soils strength characterization is not an easy task to estimate from in situ tests, due to its cohesive-frictional nature (Viana da Fonseca & Coutinho, 2008). Having two components - friction and cohesion - a strength will have to be evaluated from laboratory triaxial tests over undisturbed samples, since the evaluation of strength through in situ tests is usually conveyed under pure frictional granular soils, under the bias of angle of shearing resistance (frictional and dilatant components) or under Tresca type shear strength when geomaterials are analysed in total stresses, as it is the case of undrained shear strength in clayey soils, or the maximum total shear strength in very hard soils and soft rocks. This will be approximate to the concept of “cohesive” soils, where the strength is mostly a property explicitly non-frictional.

In the case of residual soils, cohesive strength is related with the inter-particular bonding inherited from the parent rock that provided the cemented structure and with the contribution of suction, when this is present. On the other hand, the angle of shearing resistance comprises two portions, as stated above. One is related with the pure friction that is mobilized during the relative movement between the particles. The other concerning the resistance that is mobilized during this relative movement, required to destroy the natural inter-particular cementation and its spatial organization, that is, its fabric. It should be stressed that due to the usually low void ratio of these materials typically there is an increase in volume (positive dilatancy) during shear.

The most straightforward way to characterize this type of strength is through triaxial tests, but the process has to face the important disadvantage related with sampling disturbance (Ferreira et al., 2011), where the partial loss of cementation structure is mostly unavoidable. The referred sampling disturbance and the discontinuous information related to laboratory tests leave an important role to in-situ tests on residual soil characterization for routine analysis, especially those that induce small disturbance during installation and allow the direct estimation of stress-strain response (Viana da Fonseca et al., 2011), such as pressuremeters (self-boring pressuremeter, SBPT, in a first degree and, as a fair compromise, Ménard pressuremeter, PMT) and dilatometers (such as Marchetti Flat Dilatometer, DMT). Seismic tests may give relevant interpretation when associated to the more simple tools (seismic dilatometer SDMT, seismic piezocone SCPTu), as they give a reference value of the small strain shear modulus \( G_0 = p.V'_s \).

Some important works modelling in-situ tests in residual soils have been undergoing, such as the new cavity expansion model that incorporates the effects of structure and its degradation (Mantaras & Schnaid, 2002; Schnaid & Man...
tars, 2003), the extension of the cavity expansion theory to unsaturated soils (Schmaid & Coutinho, 2005) and the overall fitting Self-Boring Pressuremeter pressure-expansion curve (Fahey & Randolph, 1984; Viana da Fonseca & Coutinho, 2008; Topa Gomes et al., 2008; Topa Gomes, 2009). Viana da Fonseca (1996) had also highlighted the utility of plate load tests, by performing series of tests with different plate sizes allowing the determination of strength parameters (c’ and φ’) by multiple optimization of the results, although this procedure is time-consuming and the limitation of involving very superficial horizons makes it less attractive.

2. DMT Tests in Residual Soils

DMT tests can be seen as a combination of some features of both CPT/CPTu and PMT tests with some details that really make it a very interesting technique available in modern geotechnical characterization. The fundamental advantages of the test are related with the high level of precision measuring both the pressures and the displacements, the response supported by semi-spherical expansion theories, the quasi-continuous profiles that provide a reasonable amount of data adequate for statistical analysis, the numerical identification of soil type, the deduction of intermediate parameters that represent common geotechnical features (namely deformability and stress history) and its easy combination with any type of in-situ and laboratory tests.

In its essence, dilatometer is a stainless steel flat blade (14 mm thick, 95 mm wide and 220 mm length) with a flexible steel membrane (60 mm in diameter) on one of its faces. The equipment is pushed (or driven) into the ground, by means of a CPT rig or similar, and an expansion of the membrane is conventionally performed every 20 cm depth. At each depth, the penetration is stopped and the membrane is expanded to lift-off pressure (\( p\_0 \) - \( u\_0 \), where \( u\_0 \) is the equilibrium pore pressure) normalized by the in-situ effective vertical stress. The parameter can be regarded as a \( K\_1 \) amplified by penetration, with normally consolidated (NC) deposits with no ageing and/or cementation structure represented by the value \( K\_1 \approx 2 \) (in clays) (Marchetti, 1980). Furthermore, the typical \( K\_1 \) profile is very similar in shape to the OCR profile giving useful information not only about stress history but also on the presence of cementation structures (Cruz et al., 2004a; Cruz 2010). In general the evolution of \( K\_1 \) profiles follows some typical trends, (Marchetti, 1980):

a) \( K\_1 \) profiles tend to follow the classical shape of the OCR profile;

b) Normally-consolidated (NC) soils tend to present values of \( K\_1 \) around 2;

c) Over-consolidated (OC) soils show values of \( K\_1 \) above 2, decreasing with depth and converging to NC values;

d) Normally consolidated soils affected by cementation or ageing structures show values of \( K\_1 \) higher than 2, remaining fairly stable with depth.

The theory of elasticity is used to derive the dilatometer modulus, \( E\_d \) (Marchetti, 1980), by considering that membrane expansion into the surrounding soil can be associated to the loading of a flexible circular area of an elastic half-space, from which the outward movement of the membrane centre under a normal pressure variation, \( \Delta p = p\_1 - p\_0 \), can be calculated. Considering the characteristics of the test, the dilatometer modulus is represented by the equation:

\[
E\_d = \frac{E}{1 - \nu^2} = 34.7 \Delta p
\]

where \( E \) represents the Young’s modulus and represents the Poisson’s ratio.

At last, the pore pressure index, \( U\_0 \), (Lutenegger & Kabir, 1988) is related to pore pressure condition, which is quite similar to \( B\_1 \) of CPTu tests.

DMT tests have been mostly used in sedimentary soils, where the test has shown its remarkable usefulness in estimating stress history, strength and stiffness characteristics in soft/loose to medium soils. However, the application
of the test to residual soils characterization is still begin-
ning, mostly within research programs. In general practice,
is rather common to see applying correlations developed
for sedimentary to residual soils characterization, which
frequently leads to erroneous interpretations. In the case of
Portuguese residual sandy soils, the angle of shearing resis-
tance is clearly overestimated because it incorporates the
cohesive component of strength due to cementation, while
this (cohesive) parcel cannot be calculated since is not con-
sidered in sedimentary approaches for the test interpreta-
tions. Since at least two basic parameters ($p_c$ and $p_f$) are
obtained, DMT offers a good via to separate cohesive from
friction/dilatancy components.

Previous research using DMT alone or combined
with CPTu tests in residual soil characterization (Cruz,
1995; Cruz et al., 1997, 2004a, b; Cruz & Viana da Fon-
seca, 2006a, 2006b) has shown that the test could be effi-
ciently used to derive strength parameters of residual soils
from Porto Granites. Firstly, Cruz et al. (1997) based their
hypotheses on two well documented cases (Cruz, 1995;
Viana da Fonseca, 1996) following the proposal from Mar-
chetti (1980) from which $K_o$ may be a well adapted index
parameter for detecting cementation and quantify its mag-
nitude. The results showed that typical profiles were more
or less stable with depth, within 5-7 interval, indicating cer-
tement or ageing according to Marchetti’s (1980) initial state-
ments. However, the correlation with the values of the
cohesive intercept obtained from triaxial testing in undis-
turbed samples revealed poor sensitivity of the parameter to
be used with success. This is certainly related to the fact
that $K_o$ depends only on $p_o$, which is the parameter more af-
fected by the blade penetration, so mostly associated to the
horizontal stress state, more than natural interparticle ce-
m entation.

A second step was attempted by Cruz et al. (2004b)
and Cruz & Viana da Fonseca (2006a), based on the OCR
parameter derived from DMT. In fact, OCR correlates
strongly with the range between $p_o$ and $p_c$, and is an amplifi-
cation of $K_o$, thus bringing more sensitivity to small varia-
tions of cohesive intercept. Although the concept of over-
consolidation does not have a meaning in residual soils and
is very questionable in the case of sandy soils, the presence
of a naturally cemented structure gives rise to some aspects
of the mechanical behaviour, similar to those observed in over-
consolidated clays, as it is sustained by Leroueil & Va-
ughan (1990). In short, the behaviour of a cohesive-
frictional soil will show an important variation in the stress-
strain behaviour when the cementation bonds start to
break, which will be very similar to the response of over-
consolidated soils, although with quite distinct pattern of
the relative values of void ratios, namely under the classical
critical state approaches (Viana da Fonseca et al., 2011). In
the case of cohesive-frictional materials, the pre-consoli-
dation stress as defined in sedimentary clays would be di-
rectly associated to the magnitude of the cohesive compo-
nent of strength related mainly to interparticle cementation
and/or suction, but also to some contribution to dilatancy.
For this reason, the concept is usually designated as “vir-
tual”, $v$OCR (Viana da Fonseca, 1996), or “apparent” over-
consolidation, AOCR (Mayne, 2006), since it does not rep-
resent a pre-consolidation stress as in the true OCR, but
only the strength arising from structural interparticle ce-
m entation. Having in mind that residual soils from Porto
Granite are mostly sandy silts to silty sands, OCR derived
from DMT in granular sedimentary soils (Marchetti
& Crapps, 1981) was selected as reference parameter for cor-
relation with the cohesive intercept in Mohr-Coulomb
strength criteria. In parallel, the ratio between constrained
modulus ($M_C$) and CPTu corrected tip resistance ($q_t$) was
also studied as an index parameter, because it is also related
to overconsolidation ratio in sandy soils (Baldi et al., 1988;
Jendeby, 1992) and could work as a control parameter of
OCR derived from DMT. Based on a large quantity of
in-situ information collected in Porto Granite Formation
(40 boreholes with regular Standard Penetration Tests SPT,
36 DMT, 22 CPTu, 4 PMT, 5 Dynamic Probing Super-
Heavy tests DPSH), specific correlations were proposed to
deduce the effective cohesion intercept from vOCR and
$M_C/q_t$. The obtained data clearly demonstrated parallel
trends followed by both indexes and a similar good level of
accuracy of the respective correlations to deduce cohesive
component of strength (Cruz et al., 2004a; Cruz & Viana da
Fonseca, 2006a). Since they are independent evaluations,
this convergence seems to confirm the adequacy of the pro-
posed approach.

The presence of a cemented structure also creates a
serious obstacle to derive angles of shearing resistance from
in-situ tests (SPT, CPTu, PMT and, of course, DMT)
when sedimentary soil correlations are used, because they
were developed on the principle of a (unique) granular
strength (Viana da Fonseca, 1996; Cruz et al., 2004a; Cruz
& Viana da Fonseca, 2006a; Viana da Fonseca et al., 2009;
Cruz, 2010). Resistance in cemented soils is then “ass-
sumed” in the same way as in granular materials, that is
high values of strength associated to higher values of the
angle of shearing resistance (peak value) incorporating the
frictional component of a critical state value, plus dilat-
tancy. In short, cohesive component of strength is “trans-
formed” in an “equivalent” additional value on the angle of
shearing resistance and once the cohesive intercept is ob-
tained, it is reasonable to expect that it can be used to cor-
rect the over-estimation of that angle, derived from sedi-
mentary correlations. Considering the low influence that
sampling has on the evaluation of angle of shearing resis-
tance (Viana da Fonseca, 2003), the observed difference
between calculated $\varphi_{\text{DMT}}$ (which represents the global
strength) and $\varphi_{\text{MDMT}}$ (which represents solely the friction
plus dilatants value) should correlate with the magnitude of
cohesion, that is with vOCR (Cruz & Viana da Fonseca,
2006a).
Although the proposed correlations were established with careful triaxial testing programs, the results of reference cohesion were obviously affected in an unknown extent by sampling disturbance and space variability, and therefore the reference values used to define these correlations would be deviated from “in-situ” real conditions (Cruz et al., 2004a; Cruz, 2010).

3. Experimental Framework

As a consequence of the effects of sampling on the derived correlations referred in the previous section, it became fundamental to develop an experimental programme in controlled conditions that could avoid those effects to settle more definitive correlations. To do so, a special large dimension container with diverse measuring systems (CemSoil Box) was created in order to work with large artificially cemented samples where DMT blades could be installed, remoulded in the same conditions as those that would be tested in triaxial apparatus. Moreover, it was also decided to pre-install blades, aiming to evaluate the static penetration influence in the loss of cementation strength, and the overall effects on stiffness. The whole experience (Cruz, 2010) relied on residual soils from Guarda Granitic Formation, characterized by patterns of behaviour identical to those observed in Porto Granite Formation (Viana da Fonseca, 1996, 2003), where the previous research (Cruz et al., 2004a; Cruz & Viana da Fonseca, 2006a) had been performed. The natural spot from where the soil was collected is located in a very well characterized experimental site (Rodrigues, 2003), investigated by laboratory triaxial tests (within the same ranges of confining stresses and void ratios of the present experiment) and in-situ CPTu, PMT and DMT tests.

Four different compositions of soil-cement mixtures and one uncemented were prepared to be tested in CemSoil, followed by an exhaustive laboratory program, including uniaxial, tensile and triaxial testing at low to medium confining stresses, performed on samples prepared in the same conditions of CemSoil. As a consequence of these dispositions, it was possible to create comparable controlled conditions, namely in curing times, compaction procedures, final unit weights and void ratios. Therefore, the sampling problems were avoided and the effects of DMT blade insertion on naturally cemented residual soils could be reproduced and studied, aiming to correct the empirical correlations previously proposed by Cruz et al. (2004b). In summary, the most relevant issues considered in the preparation of the experiment were the following:

a) Artificially remoulded samples were used in the present experiment avoiding problems associated to sampling damage, by eliminating this dispersion factor;
b) CemSoil and triaxial samples had the same curing times, void ratios (or unit weights) and cementation levels when tested, thus space variability and microfabric differences were tentatively minimized;
c) Since CemSoil samples were prepared in one time with the water being introduced only after curing, then observed differences in strength above and below water level should be mainly due to suction effects;
d) The testing sequences in Cemsoil box were controlled by using piezometers, tensiometers and geophones for seismic wave velocities measurements;
e) The experiment was based in DMT measurements both in pre-installed and pushed-in blades, which allowed to compare the influence of its penetration.

3.1. Laboratory calibration program

As previously referred, four different compositions of soil-cement mixtures and one uncemented were prepared to be tested in CemSoil Box and in an exhaustive laboratory program. This program included uniaxial and diametral compressive tests performed in saturated and unsaturated conditions, as well as isotropically consolidated drained (CID) triaxial testing at low to medium confining stresses (25, 50, 75 and 300 kPa), in saturated conditions. On the whole, the laboratory program included 40 unconfined, diametral and triaxial compressive tests. Figure 1 illustrates some details of triaxial testing system.

Figure 1 - Triaxial testing: a) Artificially cemented sample; b) LVDT installation; c) Test apparatus.
The mixtures were moulded to represent the geotechnical units existing in Porto and Guarda Granites, corresponding to identical ranges of uniaxial and tensile strengths, both of them used as cementation reference indexes. This indexation was supported by the extensive data collected for Porto Geotechnical Map (COBA, 2003) from which a global study on the mechanical degradation with weathering of Porto Granites was made (Cruz, 2010). Guarda Granites geotechnical information (Rodrigues, 2003) was compared and fitted within that data base.

Two types of cement have been used, namely SECIL CIM I/52.5R (Mixtures 1 and 3, corresponding respectively to 1% and 2% of cement) and CIMPOR CIM II/B-L 32.5N (Mixtures 2 and 4 corresponding respectively to 2% and 3% of cement). Detailed discussion about this combined use can be found in Cruz (2010), but it should be mentioned that the whole research program was based in considering exactly the same curing time for each pair of samples (laboratory and the corresponding CemSoil samples). Compressive and tensile strength were used for indexation, instead the percentage of cement. Therefore, as far as the accuracy of these index parameters and the similarity of CemSoil and laboratory samples were ensured, the combination of the both cement types could be considered acceptable.

3.2. CemSoil box

CemSoil box (Fig. 2) is a container (Large box) with 1.5 m height steel box with a square cross section of 1.0 m$^2$, with 3 mm thick steel walls, reinforced by metal bars placed at 1/3 and 2/3 of its height. Each panel was fixed to the adjacent with a profile of 5 screws (10 mm) with 150 mm of influence radius. Due to the panel-to-panel fixation system, in two of the faces this reinforcement system was in contact with the wall by a central 7 mm thick H beam (100 x 50 mm$^2$) placed vertically. This system aimed to reduce horizontal displacements during compaction processes. The inner surfaces (vertical walls) and bottom surface of the cell were covered with a plastic film, in contact with the steel wall, followed by 15 mm Styrofoam plates in order to create a smooth and flexible transition between the soil and the external border.

Figure 3 shows a plan view and a vertical cross-section of CemSoil box with the distribution of installed equipment. CemSoil block samples (1.0 x 1.0 x 1.5 m$^3$) were compacted in homogeneous layers of 70-80 mm, aiming to create homogeneous samples, with similar void ratios in CemSoil and triaxial testing, thus creating comparable conditions. The compaction in CemSoil box was handmade, using a round wood hammer of 40 cm diameter.

Considering the main objectives of the experiment, two DMT blades were positioned during the compaction processes, one being placed 20 cm above CemSoil base level and the other 25 cm below the surface upper level of the soil. Furthermore, since residual soils are commonly affected by suction phenomena, which clearly affect their strength and stiffness behaviour and mislead the interpretation of in-situ test results, block samples were only partially saturated to have the chance of studying the influence of suction on DMT results. For this purpose, two open tube PVC piezometers were installed, one located nearby the water entry in CemSoil and another in the opposite corner, which allowed to control the water level and its stabilization during the main experiment. To evaluate the generated suction profile, six tensiometers were placed at different locations in Cemsoil box. Finally, three pairs of geophones to evaluate compression ($V_p$) and shear ($V_s$) wave velocities were placed vertically and horizontally (Almeida et al., 2012), respectively for suction and low energy seismic vibrational wave velocities determinations. Figure 4 illustrates this monitoring system.

The location and distribution of all these measuring tools within CemSoil box was chosen with reference to some available published works on the subject, namely those that studied the influence of DMT dimensions by

![Figure 2 - CemSoil Box: a) CemSoil sample ready for testing; b) View after testing.](image-url)
strain path analysis (Huang, 1989; Finno, 1993; Whittle & Aubeny, 1992) or flat cavity expansion analysis (Yu et al., 1993; Smith & Houlsby, 1995). Numerical modelling of the penetration phase, using the strain path analysis (Whittle & Aubeny, 1992), pointed out some useful indications about the soil volume that may be influenced by the dilatometer insertion, which were considered in accordance.

The experiment with DMT in CemSoil box included pre-installed and pushed-in blades to analyse the effects of penetration on the final results. However, pushed-in tests were performed only in the destructured non-cemented sample and in Mixture 1 and Mixture 2, since due to its high resistance it was impossible to penetrate the equipment in the remaining mixtures. In all samples, regular measurements of suction pressures and seismic wave velocities were obtained during curing periods, before and after the saturation phase, which was concluded two days before each test. Finally, at each pre-selected testing day, DMT expansion tests of the first (below water level) and second (above water level) installed blades were made. After these tests, the second testing sequence with the blade being pushed-in down to the first blade depth was executed (details in Cruz, 2010). The respective results were then compared with triaxial test results in terms of strength and stiffness parameters, from where the correlations based in DMT parameters were developed.

Figure 3 - Vertical cross section and plan view of CemSoil instrumentation.

Figure 4 - Testing devices: a) Tensiometers b) Geophones.
4. Discussion of Results

4.1. Laboratory strength evaluation

Table 1 presents a summary of laboratory results obtained both in naturally and artificially cemented samples, indexed by the \( N_{\text{SPT}} \) (SPT blow count) ranges found in Porto and Guarda natural geotechnical units. These ranges were settled by considering the comparable ranges of uniaxial and tensile strengths found in Porto Geotechnical Map (COBA, 2003).

The reference strength parameters \( (c', \varphi') \) were obtained through CID triaxial testing, following the Mohr-Coulomb strength criterion, assuming the failure as corresponding to the maximum of the stress ratio \( q/p' \) (where \( q \) is the deviator stress and \( p' \) is the mean effective stress) mobilized during shear. The obtained failure envelopes in the artificially cemented mixtures and destructured uncemented samples are represented in Fig. 5, which clearly shows the non-linearity of the envelope, more evident with increasing cementation levels. As so, the envelopes deviate from the theoretical Mohr-Coulomb model, which assumes linearity between normal and shear stresses in the failure plane.

The aforementioned deviation should be understood as a consequence of complex phenomena that rules the shear strength mobilization in this type of cohesive-frictional materials. In fact, in order to allow the relative movement of particles, fabric interlocking creates an extra resistance during shear that commonly generates a volume increase (positive dilatancy), as a result of the usually medium to low void ratios of these soils. The higher is the interlocking, the higher will be the strength arising from this effect. On the other hand, although the ratio between friction forces that are mobilized in the surface of the particles and the installed normal forces is linear, the required forces to overcome interlocking vary with the magnitude of normal forces, which aggravates the non-linearity of failure envelope. Figure 6 gives a closer look of the relationships between dilatancy \( (d = \delta\varepsilon/\delta\varepsilon_c, \text{where } \delta\varepsilon_c \text{ is the increment of volumetric strain and } \delta\varepsilon \text{ is increment of shear strain}) \) and stress ratio, \( \eta = q/p' \) for the conventional drained tests. In order to simplify the analysis only some of the obtained results are plotted, namely the uncemented, the weakest (Mix 1) and the strongest cemented (Mix 4) samples, subjected to lower and higher confining stresses \( (p'_0 = 25 \text{ kPa} \text{ and } p'_0 = 300 \text{ kPa}) \). The results show that for the uncemented sample the relationship \( d_q/p' \) is essentially linear, showing that the mobilized resistance is eminently frictional, while in cemented materials the increment of effective confining stress leads to a decreasing dilatancy and generates a higher punctual friction between particles as a result of the normal stress increase. Figure 6a reveals that there is a volume increase in shearing, being the strain levels related with maximum stress ratio \( (q/p') \) and maximum dilatancy very close.

Another characteristic highlighted by Fig. 6 is that the cementation structure is increasingly degraded with increasing confining stresses. In fact, the stress-dilatancy behaviour of Mix 4 (25 kPa) reveals a first part with an increasing volume reduction up to the coordinates \( [d = 0.7; q/p' = 0.9] \), which might be related with the readjustment to the initial conditions of the test. From this point on, the evolution is almost vertical up to the coordinates \( [d = 0.82; q/p' = 2.1] \), meaning that dilatancy remains fairly constant and corresponds to an elastic response of the soil. This suggests that cementation opposes to the volume increase. With the stress evolution, the cemented structure starts to

![Figure 5 - Failure envelopes from triaxial tests performed in artificially cemented soils and natural de-structured soils.](image)

### Table 1 - Laboratory test results related with CemSoil samples.

<table>
<thead>
<tr>
<th>Strength</th>
<th>Naturally cemented</th>
<th>Destructured non-cemented</th>
<th>Mixture 1</th>
<th>Mixture 2</th>
<th>Mixture 3</th>
<th>Mixture 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ranges of ( N_{\text{SPT}} ) (blows/30 cm)</td>
<td>10-30</td>
<td>-</td>
<td>10-30</td>
<td>30-60</td>
<td>&gt; 60 (ISRM W.)</td>
<td></td>
</tr>
<tr>
<td>Uniaxial compressive strength, ( q_u ) (kPa)</td>
<td>81.3</td>
<td>20.8</td>
<td>72.6</td>
<td>124.9</td>
<td>273.0</td>
<td>312.3</td>
</tr>
<tr>
<td>Diametral compressive strength, ( q_d ) (kPa)</td>
<td>12.3</td>
<td>1.5</td>
<td>7.2</td>
<td>15.3</td>
<td>33.2</td>
<td>39.4</td>
</tr>
<tr>
<td>Cohesion, ( c' ) (kPa)*</td>
<td>37.1</td>
<td>0.0</td>
<td>23.8</td>
<td>38.4</td>
<td>63.2</td>
<td>107.7</td>
</tr>
<tr>
<td>Angle of shearing resistance, ( \varphi' ) (°)*</td>
<td>34.0</td>
<td>35.0</td>
<td>33.0</td>
<td>34.0</td>
<td>30.0</td>
<td>30.0</td>
</tr>
</tbody>
</table>

*Results obtained in triaxial compression tests in saturated samples, cured in the same conditions of the set of tests, both in laboratory and in CemSoil box.
break generating a yield (Second yield, according to Coop & Willson, 2003) followed by a rapid evolution towards maximum dilatancy (close to peak resistance), when generalized failure occurs (Gross yield, according to Coop & Willson, 2003). This means that for the maximum stress ratio (assumed as the failure criterion) the strength resulting from cementation is already significantly affected. After peak strength, there is a decrease on dilatancy rates of evolution that follows the strength decrease towards the critical state (constant resistance with null dilatancy). The same sample (Mix 4), when subjected to higher confining stresses (300 kPa), shows a smoother pattern of variation with no signs of volume increase, evolving directly to critical state after generalized bond breakage. This may be related with the previous partial bond breakage during consolidation phase and also by the higher stress restraining the volume increase. Therefore, strain levels related to peak strength increase with effective confining stress, which will probably generate higher level of destructuration. The behaviour of the less cemented sample (Mix 1) is located within the behaviours of uncemented and more cemented sample (Mix 4). In conclusion, as the cementation level decreases, the stress-dilatancy behaviour approaches linearity revealing loss of importance of the cement portion.

From the practical point of view, which is at the base of the present research, this complex behaviour creates serious difficulties to establish a simple way to get mechanical parameters of these residual soils for design. Firstly, because design of foundations or retaining walls and slope stability analysis usually are based on limit equilibrium models that depend on the Mohr-Coulomb strength criterion, where the dilatancy is not an input parameter. In such case, to estimate the Mohr-Coulomb strength parameters ($c', \varphi'$) it is necessary to linearize the failure envelope. For this purpose, when wide stress fields are considered the deviations will certainly be relevant, generating an overestimation of cohesive intercept for high mean stresses and the overestimation of angles of shearing resistance in the opposite situation. However, if the stress field in the characterization programme is somewhere close to those of the geotechnical problem under study, then the deviations will have small consequences on the final results.

On the other hand, the strength contributions differentiated by cohesion, dilatancy and friction, would be very difficult to get from DMT tests, as in any other geotechnical tests used in routine analysis. In such case, triaxial and DMT results could not be easily compared, since cohesion and friction incorporate the dilatancy contributions in the case of DMT, while in the triaxial case that influence could be identified independently (Viana da Fonseca et al., 2014). As a consequence, and for simplicity, it was assumed to select comparable situations in the correlations definition, meaning that triaxial testing results were expressed in terms of a cohesive and a frictional contribution, which incorporate the influence of dilatancy and the effect of bonding that is still present when the maximum strength is mobilized. In other words, the strength parameters obtained by DMT tests from the correlations arising from this work can be directly applied in a Mohr-Coulomb based analysis, without losing the distinct contributions of the cohesive and frictional/dilatant strength that really will be present in the field.

4.2. DMT test results

4.2.1. Basic and intermediate parameters

From the basic pressures point of view, the comparison of pushed-in and pre-installed blades showed that penetration generates different disturbance consequences in non-cemented and cemented soils. In fact, in non-cemented soils and Rocks, São Paulo, 37(3): 195-209, September-December, 2014.
soils the basic DMT parameters ($p_0$ and $p_1$) are higher in the case of pushed-in tests, revealing a somehow expected effect of densification around the blade, especially on the membrane where measurements are made. On the other hand, in cemented soil mixtures the same insertion procedure reduces the values of pressures monitored during the tests by local destructuration or interparticle debonding. Accordingly, under pushed-in conditions, $p_0$ and $p_1$ increase with cementing level (Fig. 7), confirming the previously mentioned sensitivity of DMT to cementation (Cruz et al., 2004a). As a consequence of these trends, the intermediate parameters $E_D$ and $K_D$ increase with cementation, while $I_D$ remains essentially the same (Cruz, 2010).

Aiming at an effective control of the experiment with external reference, DMT tests were also performed in the same spot of natural residual soils from where the soil was collected and remoulded. The obtained results revealed that local DMT basic and intermediate parameters were situated between those obtained in Mixtures 1 and 2, which is coherent with the results of uniaxial, diametral and triaxial (cohesive) strengths previously presented in Table 1.

4.2.2. The influence of suction on DMT parameters

The results obtained in unsaturated conditions revealed an increment of the global strength and stiffness, which must be related to the presence of suction. In fact, when unsaturated results are normalized in relation to pushed-in saturated values (Fig. 8), here designated by $p_1^*$, $E_D^*$, $I_D^*$, $K_D^*$, a decrease of the magnitude of each normalized parameter when approaching the water level is observed suggesting an adequate sensitivity of DMT to detect the effect of suction, somehow expected since DMT pressures are directly measured.

This conclusion is confirmed also by the non-cemented sample results in unsaturated conditions, considering that in this case the results should reflect suction alone. Data also reveal a decay of $K_D$ and $E_D$ with increasing cementation, apart from suction influence, which is explained by the clear increase of cohesion intercept, while suction remains essentially the same. Therefore, these results clearly put in evidence the high sensitivity of DMT to both suction and cementation in the cohesive component of strength.

A different behaviour is followed by $I_D$, revealing to be mostly independent from saturation levels in cemented soils, while in non-cemented samples suction influences notoriously the magnitude of the parameter. This is in accordance with recent evidences of such behaviour, pub-
lished in Arroyo et al. (2013). In saturated conditions, the evaluation of soil type was found very accurate when compared with elemental soil classifications based on the percentage of clay, silt and sand, while unit weights obtained through \( I_o \) and \( E_o \) (Marchetti & Crapps, 1981) were confirmed by laboratory determinations on good quality extracted samples. Since the homogeneity in the large prototype (CemSoil) was carefully assessed, and differences between triaxial and CemSoil samples were minimized by the careful correspondence in the preparation of both void, moisture and cementation conditions, the above considerations strongly suggest that DMT is adequate to infer the cohesion components arising from suction and cementation, even after the blade penetration.

4.2.3. The virtual overconsolidation ratio (vOCR) and global cohesion (\( c'_g \))

As previously stated by Cruz et al. (2004a) and Cruz & Viana da Fonseca (2006a), the DMT key parameter for evaluating the cohesive strength is the virtual overconsolidation ratio (vOCR) that represents the order of magnitude at which the stress-strain behaviour changes, expressed by the enlargement of yield locus due to cementation structure. The determination of vOCR follows the same formulations proposed by Marchetti & Crapps (1981) for sedimentary soils, as follows:

\[
I_o < 1.2 \text{ (cohesive soils) } OCR = (0.5 K_o)^{1.56} \tag{2}
\]

\[
I_o > 2 \text{ (sandy soils) } OCR = (0.67 K_o)^{1.91} \tag{3}
\]

\[
1.2 < I_o < 2 \text{ (mixed soils) } OCR = (m K_o)^n \tag{4}
\]

where

\[
m = 0.5 + 0.17 P \tag{5}
\]

\[
n = 1.56 + 0.35 P \tag{6}
\]

with

\[
P = (I_o - 1.2) / 0.8 \tag{7}
\]

The considerations presented in the previous subchapter revealed that above the water level the magnitude of the DMT intermediate parameters is clearly influenced by suction, and so will be vOCR. Considering the homogeneity and similarity of the triaxial and CemSoil samples, triaxial cohesion intercept can be assumed as representative of the whole sample moulded in CemSoil. Therefore the increase in the results obtained above the water level should be interpreted as a consequence of suction. If that is accepted, a global cohesive component (\( c'_g \)) due to both interparticular bonding and suction (when the latter is present), should be the reference parameter to correlate with DMT results.

The suction measurements taken during the CemSoil experiment make possible evaluating suction contribution to shear strength, from the third term in the model proposed by Fredlund et al. (1978):

\[
\tau = c' + (\sigma - u_s) \tan \phi_s + (u_a - u_s) \tan \phi_h \tag{8}
\]

where \( u_s \) is the atmospheric pressure, \( u_a \) is the pore water pressure and \( \phi_h \) is the “angle” of increase of cohesion with suction (similar to the concept of angle of shearing resistance in its dependence on stress).

The term \((u_a - u_s)\) corresponds to the suction measured on tensiometers, while for \( \phi_h \), a 14° reference value was obtained by Topa Gomes (2009) in Porto highly weathered granites (\( W_1 \) to \( W_5 \)) and in the residual soil of ISC'2-CEFEUP (Arroyo et al., 2013), which was assumed in this analysis given the similarity of Guarda and Porto granites, in what concerns to mineralogy, grain size distribution, plasticity and solids unit weight.

Expressing the global cohesion \( (c'_g) \) results as function of vOCR, a specific correlation to derive the global cohesive intercept is obtained, as presented in Eq. 9 and in Fig. 9. In this figure, the previous correlation presented by Cruz et al. (2004a) based on triaxial testing is also represented.

\[
c'_g = 7.716 \ln (\text{vOCR}) + 2.9639 \tag{9}
\]

It is important to note that the previous correlation was based on a narrower band of vOCR values and it was defined as a straight line, while the new data are better represented by a logarithmic function. Considering this new approach, the former data were incorporated and a new match fundamental function was defined. As it would be expected, the effect of sampling processes usually leads to a reduction of cohesion intercept, with an extent that seems to be dependent on the most appropriate equipment and convenient procedures. In this case, the differences between correlations lead to a general loss (mainly related to sampling) of around one third (1/3). For future interpretations, the previous correlations were based on statically pushed-in 70 mm Shelby tube samples.

Since the parameter \( \phi_h \) is not usually available, thus it has to be estimated, it is important to analyse their influence on the final results. Taking into account the usual values for

![Figure 9 - Correlations of global cohesion intercept (\( c'_g \)) as a function of vOCR.](image-url)
this parameter in the studies developed in Porto granite residual soils, as those reported in Topa Gomes et al. (2008), and in other international reference works (Futai, 1999), a variation of 5° around the considered value was found to be sufficient. Observed deviations resulting from variations of φ within 10° and 20° are insignificant, as shown in Fig. 10.

The evolution of the global cohesion intercept, \( c'_g \), in CemSoil and in-situ test results obtained from direct application of the proposed correlation is presented in Figs. 11a and 11b, respectively. Figure 11a reveals once more the same trends observed in all other analysed parameters, with in-situ values falling between Mixtures 1 and 2, while the in-situ profile (Fig. 11b) shows a general decrease of the overall cohesion intercept until the water level is reached, remaining fairly constant after that depth with slightly lower values than those obtained in triaxial testing. It is also worth to note the convergence with the \( c'_g \) profile represented in the same figure, obtained by considering a theoretical linear evolution of suction as a function of the distance from the water level.

4.2.4. Corrected angle of shear resistance

Angles of shearing resistance in these residual soils can be derived by the approach proposed by Marchetti (1997) for sedimentary sandy soils, applying a correction factor that should be function of the result of cohesion intercept or the index DMT parameter used in cohesion correlation (vOCR), as proposed by Cruz et al. (2004a) and Cruz & Viana da Fonseca (2006a). The correlation for correcting the angle of shearing resistance, derived from the available data in the course of this framework, is presented in Fig. 12, where \( \varphi_{\text{corr}} \) represents the angle of shearing resistance obtained from the correlations applied to sedimentary soils and \( \varphi_{\text{ref}} \) is the reference angle of shearing resistance obtained by triaxial tests on remoulded conditions. As a consequence, the corrected angle of shearing resistance (\( \varphi_{\text{corr}} \)) derived from DMT can be obtained as follows:

\[
\varphi_{\text{corr}} = \varphi_{\text{ref}} - 3.3483 \ln(\text{OCR}) + 5.4367
\]  

(10)

Using this correction, the CemSoil box (pushed-in tests) and in-situ test results are compared with the respective triaxial data, revealing a good reproduction of the experimental results. Figure 13 shows that CemSoil saturated results converge well with triaxial tests, while in-situ data slightly decrease with depth due to suction effects.

4.3. Procedure to evaluate strength parameters

As a consequence of this calibration work, it was possible to propose a procedure (Fig. 14), to derive strength parameters of Porto and Guarda granitic residual soils from DMT data. Its application in other granitic environments should be verified, although the authors believe that this method may be generalized in other granitic residual sapro-

Figure 10 - Upper and lower expected limits for the cohesive intercept (\( c'_g \)) correlations.

Figure 11 - Global cohesion intercept (\( c'_g \)) results in: a) Cemsoil; b) In-situ.

Figure 12 - Correction factor for evaluating angle of shearing resistance.
litic soils. For applications in other residual geomaterials (as for instance, in lateritic soils) with different genesis or different mineralogical/chemical compositions, specific correlations should be calibrated by tentatively using the same procedure followed in this work.

The procedure to get the parameters from DMT tests starts with the evaluation of global cohesion through the correspondent correlation. If the conditions are saturated, then the result represents both global and true cohesion, since suction is null. In the case of unsaturated conditions, the global cohesion incorporates both contributions and so suction related parameters \((u_a - u_w, \phi^b)\) are required for respective differentiation. An alternative way that may be helpful, when values of these parameters (measured or estimated) are not available, is to consider the mean value of cohesion obtained in the first results below water level \((c'_\text{avg_below})\) as representative of the soil cohesion above water level. Considering this approach, it becomes possible to evaluate suction by subtracting the value of \(c'_\text{avg_below}\) from the value of the global cohesion \((c'_g)\). Finally, the corrected angles of shearing resistance \((\phi^\text{corr})\) are obtained by subtracting the correction factor (Fig. 12, in previous sub-section) to the angles of friction obtained by traditional sedimentary approach \((\phi_{DMT})\).

![Figure 13 - Triaxial and deduced angles of shearing resistance in: a) Cemsoil; b) In-situ.](image)

![Step 1 diagram - Evaluation of strength parameters in residual soils.](diagram)

\[
\begin{align*}
\text{Step 1} & : \quad \text{Evaluate global cohesion } c'_g \\
\text{Above water level} & : \quad c'_g = 7.716 \ln(\text{OCR}) + 2.964 \quad (1) \\
\text{Below water level} & : \quad c'_g = c'_\text{avg_below} \\
\text{Measurement or estimation of suction } (u_a - u_w) & : \quad c'_g = c'_\text{avg_below} \\
\text{No measurement or estimation of suction } (u_a - u_w) & : \quad (u_a - u_w) = [c'_g - c'_\text{avg_below}] / \tan \phi^b  \\
\phi^b & : \text{assumed from local experience} \\
\text{Step 2} & : \quad \text{Correct angle of shearing resistance, } \phi^\text{corr} \\
\phi^\text{corr} & : \phi_{DMT} - 3.35 \ln(\text{OCR}) + 5.4  \\
\phi_{DMT} & : \text{(Marchetti, 1977)} \\
\text{OCR} & : \text{(OCR from Marchetti & crapps, 1981)}
\end{align*}
\]
5. Conclusions

In the context of this framework a model to interpret DMT results in residual soils from Portuguese granites was elaborated, followed by an experiment in controlled conditions to establish adequate correlations with the geotechnical strength parameters, namely the global cohesion intercept (arising from the cementation structure and suction) and the angles of shearing resistance corresponding to a linearized Mohr-Coulomb failure envelope. The obtained correlations were then applied to DMT tests performed in the IPG experimental site (Rodrigues, 2003; Cruz, 2010) and the consequent results showed an excellent convergence with the reference parameters, proving to be adequate for Porto and Guarda granites.

Previous correlations to obtain strength of the same materials (Cruz et al., 2004a; Cruz & Viana da Fonseca, 2006) were affected by sampling disturbance and space variability of the triaxial testing that served as reference and did not take into account suction effects on cohesive intercept. To overcome these uncertainties a dedicated experimental facility was planned and realized, consisting of a large dimension prototype (CemSoil box), where artificially cemented soils were moulded with the purpose of executing DMT tests with pre-installed and pushed-in blades. Specimens for triaxial testing were also prepared following the same moulding process and state conditions. The obtained results confirmed that DMT parameters are influenced by both cementation and suction. Using as reference parameter a concept similar to overconsolidation ratio, designated by “virtual” (vOCR), it was possible to establish calibrated correlations for deriving a global cohesive intercept (c’g) generated by cementation and suction effects. Moreover, when suction parameters (ϕv, u)v and oun were available, the cohesion resulting from cementation structure is easily obtained by subtracting the suction contribution, calculated by the third term of Fredlund et al. (1978) strength criterion. In cases where there is no information about suction, a novel procedure to separate cementation and suction contributions was also proposed. The experiment also proved that angles of shearing resistance could be derived with accuracy by the approach proposed by Marchetti (1997) for sedimentary sandy soils, as far as a correction factor based on vOCR is used. DMT tests performed in the same spot of Guarda Granitic Formation from where the artificial samples of the main experiment were moulded, showed high convergence with triaxial results, confirming the adequacy of the correlations developed in this framework for deriving the in-situ strength parameters. The presented correlations were tested only in Portuguese granites, and so its extension to other residual soils with different genesis or different mineralogical/chemical compositions should be confirmed in the first place. The presented work may represent a path to follow.

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References


**List of Symbols**

- $c'$: cohesive intercept in Mohr-Coulomb criterion
- $c^*$: global cohesion due to cementation and suction in Mohr-Coulomb criterion
- CID: triaxial test, isotropically consolidated drained
- CPT: cone penetration test
- CPTu: piezocene test
\( d \): rate of dilatancy = \( \frac{\delta e_v}{\delta e_s} \), where \( \delta e_v \) is the increment of volumetric strain and \( \delta e_s \) is increment of shear strain.

- **DMT**: Marchetti flat dilatometer test
- **DPSH**: dynamic probing super-heavy test
- \( E_d^* \): dilatometer modulus (DMT)
- \( E_o^* \): ratio between unsaturated/saturated values of \( E_d \)
- \( I_d^* \): material index (DMT)
- \( K_d^* \): horizontal stress index (DMT)
- \( K_o^* \): ratio between unsaturated/saturated values of \( K_o \)
- **NC**: normally consolidated soil
- \( N_{	ext{NC}} \): number of blows to penetrate 30 cm in SPT
- **OCR**: overconsolidation ratio
- \( p' \): mean effective stress
- \( p_0 \): DMT first pressure reading (lift-off)
- \( p_1 \): DMT second pressure reading (membrane expansion)
- \( p_2 \): DMT third pressure reading (closing pressure)
- \( p_a \): atmospheric pressure (101.3 kPa)
- **PMT**: Ménard pressuremeter test
- \( q \): deviator stress
- \( q_c \): cone tip resistance (CPT/CPTu)
- \( q_d \): diametral compression strength
- \( q_t \): corrected cone tip resistance (CPTu)
- \( q_u \): uniaxial compression strength
- \( q_u^* \): ratio between unsaturated/saturated values of \( q_u \)
- **SCPTu**: seismic piezocone test
- **SDMT**: seismic dilatometer test
- **SPT**: standard penetration test
- \( u, u_r \): pore water pressure
- \( u_a \): at rest pore water pressure
- \( u_p \): pore air pressure
- \( U_d^* \): pore pressure index (DMT)
- \( U_o^* \): ratio between unsaturated/saturated values of \( U_o \)
- \( \psi \): effective angle of shearing resistance
- \( \phi_{\text{DMT}} \): angle of shearing resistance derived from DMT
- \( \phi_{\text{OC}} \): reference angle of shearing resistance
- \( \eta \): stress ratio \( (q/p') \).