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Effect of soil structuring on stiffness evaluated by triaxial and seismic flat dilatometer tests

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ABSTRACT

Granitic residual soils are considered structured soils. The bonding between the particles and the fabric inherited by the original rock play an important role in controlling the mechanical behaviour. These materials considered as “cemented” soils are often seen as non-textbook materials, since they do not fit into the usual behaviour of transported soils in the light of classical Soil Mechanics theories. To study the stiffness of the granitic residual soils located in Guarda (northeast of Portugal) several SDMT tests carried out on structured soils (natural granitic residual soils and artificial cemented soils) and triaxial tests (carry out with internal instrumentation), performed on samples collected at the same depth, at similar confining stresses, to those involved in the SDMT tests. The results show that in the stiffness decay curve the value of the stiffness modulus (E_{DMT} or G_{DMT}) increase with the increasing level of cementation. In addition, the strain levels involved in the calculation of the stiffness modulus (E_{DMT} or G_{DMT}) decreases with the increase of the cementation level, which can have important consequences for practical purposes of geotechnical design.

Keywords: Residual soils, cementation, stiffness decay, strain evolution.

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) by means of a restricted experimental site that includes the local residual granitic spot, a calibration chamber and an important laboratorial infrastructure. The referred program started in the late 90’s of last century with a full laboratory and in-situ characterization of the local residual granitic soil deposit, where some important findings related with residual behaviour were achieved (Rodrigues 2003).

Departing from this earlier characterization program, a new framework was developed by Cruz (2010) aiming to settled proper interpretations in these residual soils in substitution of the sedimentary existing formulae, which does not work well in these soils. The referred framework was based in artificial cemented samples installed in a calibration chamber where DMT tests were performed and in triaxial cells used in the determination of the reference geotechnical parameters. The artificial samples were used because it was firstly intended to avoid both the in-situ typical variability of these soils and the existing gap between in-situ and laboratorial tests (due to sampling), which can have important impact in the cementation structure. Based on this experimental framework, a set of new correlations to obtain cohesion magnitude and angles of shearing resistance from DMT tests was successfully established (Cruz, 2010; Cruz et al., 2012; Cruz et al., 2018). Besides the strength, some important findings on the stiffness behaviour were also achieved, suggesting further experiments to explore stiffness decay curves as a mean to understand the residual behaviour (Cruz, 2010; Rodrigues et al., 2016). Regular seismic measurements were required for that goal.

As consequence, a new testing program was carried out, consisting in six sets of SDMT, SCPTu and PMT tests, as well as boreholes and sampling for triaxial tests, aiming more detailed characterization to support the studies previously performed and open new ways to residual soil characterization with other in-situ tests. The set of natural samples collected in boreholes, performed along with the other tests mean to serve as a bridge between the laboratorial results and the natural conditions. Furthermore, the SDMT tests allowed measuring regularly shear wave velocities with depth, scarcely available in the previous investigation phases. Data from triaxial, DMT and seismic tests was selected to serve the stiffness decay analysis within the purpose of this paper. Figure 1 shows the position of the SDMT tests, as well as the locations of the boreholes were samples for triaxial testing were collected.

In terms of strength, this new testing program led to the definition of a new set of correlations to obtain cohesion magnitude and angles of shearing resistance from CPTu tests (Cruz et al. 2018) and the development of DMT and CPTu soil behaviour type (SBT) charts (Cruz et al., 2021). Similar approach to develop correlations between strength parameters and PMT results is still undergoing.

Figure 1. Position of the SDMT tests and boreholes (BH).

Besides the findings related with the strength behaviour, a significant improvement was achieved within this framework, in terms of the analysis and development of characteristic stiffness decay curves related with the residual mass. Based in seismic determinations and in the working modulus derived from SDMT test results, a new logistic model was proposed by Rodrigues et al. (2020) to predict in-situ stiffness decay curves (equation 1), since the hyperbolic model proposed by Amoroso et al. (2014) for the case of sedimentary soils could not be successfully applied to these residual soils. It is important to underline that the logistic curve can be successfully applied to the case of Porto and Guarda granitic residual soils, but it was not yet validated in other residual soils (from granite or other lithologies). The referred logistic model is represented by the equation below:

$$G = \frac{a}{\left(1+e^{-b(log(\gamma)-c)}\right)} \tag{1}$$

$$a = G_0$$
$$b = -0.0004406 \times \text{vOCR} - 0.5591$$
$$c = -4.041 - 0.02774 \times G_0 + 0.03388 \times K_D$$

where G stands for the shear modulus, G_0 is the shear modulus at small strains, γ corresponds to the shear strain, K_D is the DMT horizontal stress index and vOCR is the virtual OCR that is determined by applying Marchetti & Crapps (1981) correlation to DMT results.

The usefulness of SDMT for that purpose is sustained by the possibility of having independent measurements at small (G_0) and working strain (I_D, E_D, K_D) levels, the partially preserved cementation of the soil involved in the measurement after test penetration and its sensitivity to cementation variations referred in several publications (see Rodrigues et al., 2020).

During the referred research programs, it became clear that the strain levels associated to DMT stiffness measurements differ significantly from those considered by the international community in the case of sedimentary soils (Mayne, 2001; Ishihara, 2001; Monaco et al., 2014) and also seem to vary with the increasing cementation magnitude represented by the cohesion intercept. Since DMT tests are performed considering constant displacements related with the field pressures P_0 and P_1, the results of moduli arising from the current DMT test interpretations should be looked with care, especially when selecting values for design purposes. The subject of this paper is to present and the discuss this specific detail of the observed behaviour.

2. Geotechnical background

The city of Guarda is located in a granitic mass designated as Guarda Granitic Formation, which is part of the geological complex related with the formation of “Serra da Estrela”, the highest mountain in Portugal mainland. Guarda Granitic formation is represented by a leucomesocratic granite with quartz, sodic and potassium feldspars, biotite and muscovite, as well as kaolin, sericite and chlorite as main secondary minerals (Rodrigues 2003). The incidence of wet-moderate climate in the originally exposed rock massif turns the granitic rock into a residual mass. The water level in the field under investigation vary within a submerged stage in the wet season and 2 m to 5 m depth during the summer, which creates conditions for continuous weathering of the rock massif. As the weathering proceeds, the original bonds between the grains are broken and series of intergranular voids are opened. Furthermore, feldspars and the micas become unstable with weathering evolution, leaving a network of intragranular voids formed by leaching. As consequence, quartz (stable) grains are bonded by highly weathered (unstable) grains of feldspars and micas to form a solid skeleton with variable porosity. Guarda residual mass is characterized by a well graded silty sand/sandy silt with low or none plasticity and coefficients of permeability ranging from 10^{-6} to 10^{-7} m/s. The petrographic index, χ_d (Lumb,1962), which relates percentages of weathered and unweathered grains, falls within 0.27 and 0.64, reflecting the high weathering degrees of the local massif (Rodrigues 2003). Figure 2 shows some characteristics of the soil deposit used in the study.

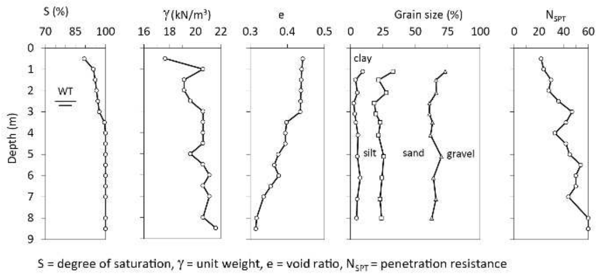

Figure 2. Soil characteristics of the massif under study.

The results of all tests showed that the local residual mass is characterized essentially by two geotechnical layers, respectively located from 1-5 m and 5-9 m depth and represented by N_{SPT} uncorrected values of 10 to 30 and 30 to 60. These two geotechnical horizons are the most frequent in the Portuguese granitic residual soils (Cruz 2010).

3. Tests performed in naturally cemented soils

3.1. SDMT tests

The seismic dilatometer (SDMT) used in the experimental work is the combination of the mechanical flat dilatometer (DMT), introduced by Marchetti (1980) with a seismic module for measuring the shear wave velocity (V_s) (Marchetti et al. 2008).

The mechanical DMT test (ASTM D6635-15 Standard Test Method for Performing the Flat Plate Dilatometer), consists in the penetration of the dilatometer blade into the ground, which is interrupted every 20 cm to apply expansion pressures to a flexible membrane. The reference pressures (A and B readings) are those necessary to expand the membrane 0.05 and 1.10 mm outwards. These A and B readings are then converted into characteristic pressures (P_0 and P_1) by correcting the effects of membrane rigidity, as follows:

$$P_0 = 1.05(A - Z_m + \Delta A) - 0.05(B - Z_m + \Delta B) \quad (2)$$

$$P_1 = B - Z_m + \Delta B \quad (3)$$

where A and B stands for the pressure to move the membrane outwards 0.05 and 1.10 mm, respectively, ΔA and ΔB are the pressures related with same displacements without the soil reaction (calibration of the rigidity of the membrane) and Z_m is the zero pressure reading in the pressure gauge.

Departing from these readings, three intermediate parameters are developed (material index, horizontal stress index and dilatometric modulus), which are then used to derive common geotechnical parameters. Material index, I_D, is related with the type of soil under analysis, horizontal stress index, K_D, is related with strength, stress history and in-situ stress state, while dilatometric, E_D, modulus is related with stress-strain behaviour. In what concerns to the latter, Marchetti (1980) defined it as follows:

$$E_D = 34.7(P_1 - P_0) \quad (4)$$

The evaluation of deformability moduli can be obtained by the equations 5, 6 and 7:

$$M_{DMT} = RmE_D \quad (5)$$

$$E_{DMT} = \frac{M_{DMT}(1+\nu)(1-2\nu)}{(1-\nu)} \quad (6)$$

$$G_{DMT} = \frac{M_{DMT}}{2(1-\nu)/(1-2\nu)} \quad (7)$$

where M_{DMT} stands for the constrained modulus, Rm is a correction factor that depends on I_D (type of soil) and K_D (stress history), ν is the coefficient of Poisson, while E_{DMT} and G_{DMT} represent respectively the Young modulus and strain modulus, which are derived from M_{DMT} through the Theory of Elasticity.

The SDMT system incorporates a cylindrical seismic module above the DMT blade, with two receivers spaced 0.50 m apart, and works as a down-hole survey according to ASTM D7400-19 Standard Test Methods for Downhole Seismic Testing. The shear modulus at small strains (G_0), is obtained from the seismic measurements using Equation 8:

$$G_0 = \rho(V_s)^2 \quad (8)$$

where ρ stands for density (ρ = γ/g; γ = unit weight, g = gravity) and V_s is the shear wave velocity.

The SDMT results obtained at Guarda site are summarized in Figure 3.

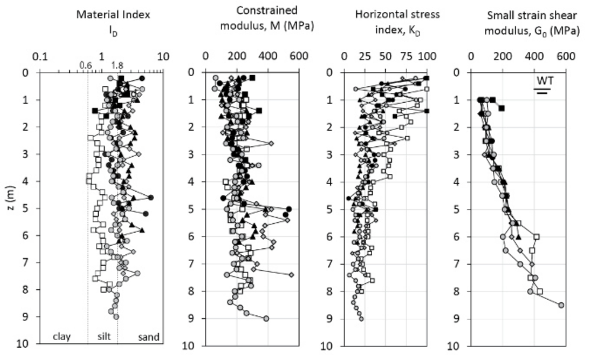

Figure 3. SDMT results in natural soils.

From the soil identification point of view, the material index I_D is fully consistent with the typical grain size of these granitic residual soils, confirming the adequacy of the parameter to identify these soils (Cruz 2010). The horizontal stress index K_D ranges mostly between 10 and 40, clearly pointing out a significant cementation structure in the soil, confirmed by the particularly high values of M_{DMT} around 200 MPa in the upper 4 m depth and 325 MPa in the lower part of the profile. These different magnitudes of the modulus clearly correspond to different weathering levels that were identified in all the other performed tests. Finally, the G_0 profile shows a linear increase with the depth.

3.2. Triaxial tests

A set of three undisturbed samples was retrieved with a thin sampler specifically built for these soils (Rodrigues, 2003), from BH_1 and BH_2 boreholes, 1 m distant from $SDMT_1$ and $SDMT_6$, respectively. Specimens of these samples were prepared and subjected to triaxial tests. Natural samples were subjected to isotropic consolidation (50, 150 and 300 kPa) and sheared under drained conditions (CID tests), using local instrumentation (LVDT's) by means of a pair of axial transducers and a Bishop ring for measurement radial deformation.

The selected stress paths were similar to the ones followed by the DMT tests, which corresponds to constant σ'_3, or at a constant ratio of $\Delta q/\Delta p'=3$ in space of stresses q, p. In this case, $q=(\sigma_1-\sigma_3)$, $p'=[1/3(\sigma'_1+2\sigma'_3)]$, $\sigma'_1=\sigma'_v$ and $\sigma'_3=\sigma'_h$ with σ'_1 and σ'_3 respectively representing the minimum and the major principal effective stresses, while σ'_v and σ'_h represents respectively the vertical and the horizontal effective stresses.

Figure 4 shows the secant stiffness-strain behaviour, obtained from the three samples collected at different depths and consolidated at 50 kPa. The initial value of G_{sec} for the sample BH_2-3.4 m (Fig. 4c) is much lower than for the samples BH_1-1m (Fig. 4a) and BH_2-1m (Fig.

4b), which is certainly due to a higher level of disturbance of the sample taken at 3.4 m depth.

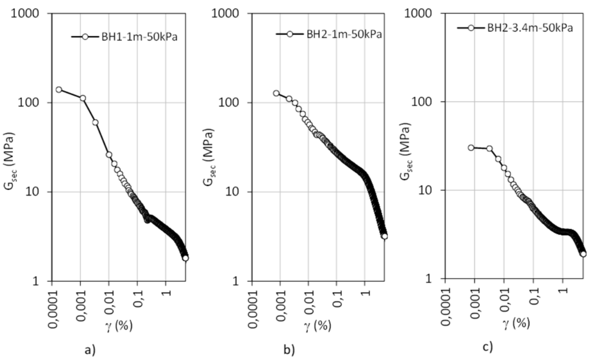

Figure 4. Soil Stiffness obtained from CID triaxial tests on natural soil samples consolidated at 50 kPa; BH_1, depth 1 m; b) BH_2, depth 1 m; c) BH_2, depth 3.4 m.

4. Tests performed in artificially cemented soils

From the earlier characterization, a special testing programme based on artificially cemented soils was established by Cruz (2010), aiming to correlate the results of flat dilatometer test (DMT) with soil strength and stiffness parameters. This experiment was carried out in a Controlled-Condition large-scale Chamber (CCC) where the testing conditions of DMT tests could be controlled. The experiment consisted in making artificially cemented samples for the chamber and triaxial tests, remoulded exactly in the same conditions to reach comparable situations. The mixtures consisted in previously de-structured residual soil (obtained in the IPG experimental site residual mass), followed by artificial cementations with different contents of Portland cement.

4.1. Sample preparation

The CCC samples were obtained by compaction of the artificial mixture, prepared in layers of 70-80 mm thick to obtain void ratios of the same order of magnitude of those observed in-situ. At the end of the process, the total height of the compacted soil corresponded to 17 layers with a total height of 1.25 m. During the compaction, two DMT blades were installed, one 20 cm above the base level, and another placed 25 cm below the surface, in the upper level of the CCC sample. To control the water level, two open tube PVC piezometers were installed, one located in one corner, from where the water was introduced at the base of the chamber, and the second located at the opposite corner to confirm the water arrival and the respective level stabilization. Above water level, suction measurements were obtained by means of six installed tensiometers. Finally, three pairs of geophones for seismic survey were placed along one profile, to measure compression and shear wave velocities. Detailed information can be found in Cruz (2010).

Samples were prepared aiming to observe the influence of cementation level in the mechanical behaviour, reproducing the in-situ conditions. Of course, reproducing the fabric and cementation level simultaneously is extremely complex to achieve, therefore samples were constituted considering always the same void ratio and varying cementation levels. In conformity, three samples for triaxial tests were prepared - one remoulded with no added cement (Rem 0) and two mixtures with different percentages of Portland cement (Mix 1 and Mix 2). Compaction (Proctor) tests were used to determine the characteristic values (optimum water content and maximum dry density) that allows the same void ratio observed in-situ.

Sample Rem 0 was composed with dry de-structured granitic residual soil and 10.5 % of water. Mix 1 was prepared with the same dry soil, which was mixed with 1% cement (CEM I 52,5R) and 10.5 % of water. Mix 2 was constituted with the dry soil, 2% cement (CEM II/B-L 32,5N) and 10.5 % of water. Table 1 presents some characteristics of the used cements.

For each level of cementation, uniaxial compressive (UCS), diametral compression and triaxial tests were performed on artificial samples remoulded and cured exactly in the same conditions of those observed in CCC samples.

The mixtures in CCC chamber were left in the curing process and at pre-designated days, water was introduced within the chamber until reaching 118 cm of depth, above the position of the lower blade. Regular measurements of suction and seismic wave velocities were acquired during the curing period, before and after saturation.

Table 1. Characteristics of the used cements

	Mix 1	Mix 2
Name	CEM I 52,5R	CEM II/B-L 32,5N
Strength class	52.5 R	32.5
Compressive strength (MPa)	30.0 (2 days)	16.0 (2 days)
	52.5 (28 days)	32.5 (28 days)
Clinker	> 95%	65% - 79%
Setting time (min)	start > 45	start > 75
Expansion (mm)	< 10	< 10

After complete stabilization of water level, DMT readings were taken on the first and second pre-installed blades, followed by the readings taken on the second blade pushed at regular intervals of 20 cm, towards the first blade testing depth. The DMT results used in the present research were those obtained in the upper blades after statically pushed down to have a situation comparable to the usual test procedures, from which the curves must be obtained. Pre-installed blade derived results were used to study and modelling the penetration influence, subject that is out of scope in the present paper.

4.2. DMT results and seismic measurements

As indicated in the previous section, three sets of geophones (vertical and horizontal) were located at three different depth locations (0.2, 0.6 and 1.2 m) of the CCC chamber in the same vertical alignment. At each test depth, two geophones were installed, one in horizontal position and one vertical, for P-wave and S-wave velocity measurement, respectively. A block of 117.6 N and an impact plate lying under rolling bars composed the source for the generation of S-waves. The load was applied on the impact plate with a good coupling between the beam and the soil, improving the quality of wave propagation. The blow generates a vibratory action with higher acceleration than the one that would have been obtained considering a fixed total mass of plate and load. This configuration creates sharper signals and higher efficiency in first arrival determination (Almeida et al. 2012). Compression and shear wave velocities were obtained after the blade installation, before and after saturation, as well as during testing time.

Figure 5 shows the material index, I_D, constrained modulus (M or M_{DMT}), and horizontal stress index (K_D), profiles obtained by DMT blades after penetration. DMT readings at 1.2 m depth are influenced by its position near the chamber base, thus should be neglected, while shear wave velocities are not affected by the low position of geophones because the velocity was calculated between that point and the surface.

The results shown in the Figure 5 indicate that the evolution of the cementation level is well represented by a global translation to the right (along with increasing cementation magnitude) of the M, K_D and G_0 profiles related with Mix 1 and Mix 2, although the observed difference between the mixture Mix 1 and the non-cemented sample (Rem0) is not so clear and sometimes with random variation. This could be explained by the development of suction that will hide the cementation influence when the magnitude is small. The results of tensile strength (q_t), frequently seen as index parameter for the level of cementation, show values of 1.5, 7.2 and 15.3 kPa respectively for Rem0, Mix1 and Mix2, seem to justify the apparent overlap of these two profiles (low value of Mix 1 tensile strength and the small difference to Rem0).

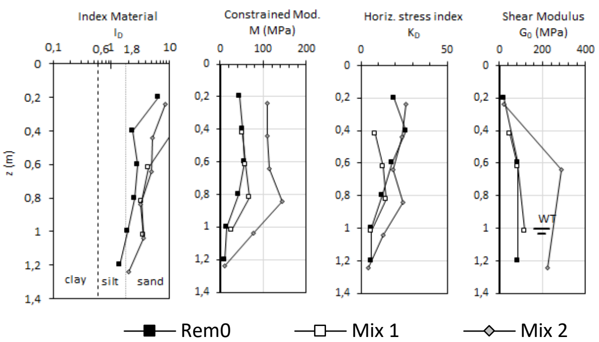

Figure 5. SDMT test results in artificially cemented soils (pushed blades).

Finally, I_D profiles show convergent values for Mix 1 and Mix 2, which in turn are higher than Rem0 profile. This is consistent with the known effect of cementation that aggregates grains with correspondent increase of the equivalent diameters.

4.3. Triaxial tests

As described in a previous section, in the sequence of the preparation of the CCC samples, identical mixtures were remoulded under the same conditions (same de-structured soil sample, percentage of cement, void ratios and curing times) to be used in triaxial testing. The preparation consisted of four layers 3.5 cm thick, statically compacted using a split mould for adequate extrusion. The samples were then stored in a curing chamber with automatic control of environmental conditions (temperature ≈ 20±1ºC and humidity ≈ 95±5%). The cementation level was referenced by uniaxial and diametral compressive strengths, instead of the percentage of cement, as these parameters are representative of the mechanical behaviour and are highly influenced by the presence of cemented structures (Cruz, 2010). Table 2 summarizes the results of uniaxial compressive strength (q_u), tensile strength (q_t) and maximum deviatoric stress (q_{max}) obtained in triaxial tests samples confined at 50 kPa.

Artificial triaxial samples were subjected to isotropic consolidation (50, 150 and 300 kPa) and sheared under drained conditions (CID tests), using local instrumentation (LVDT's) by means of a pair of axial transducers and a Bishop ring for measurement radial deformation.

The selected stress paths were similar to the ones followed by the DMT tests and triaxial tests on natural samples, which corresponds to constant σ'_3, or at a constant ratio of $\Delta q/\Delta p'=3$ in space of stresses q, p.

Table 2. Laboratorial test results in artificially cemented soils

Test	Parameter	Rem0	Mix1	Mix2
UCS	q_u (kPa)	20.8	72.9	124.9
Diametral compression	q_t (kPa)	1.5	7.2	15.3
CID triaxial tests	$q_{max}=\sigma'_1-\sigma'_3$ (kPa)	130	231	314

5. In-situ G-γ decay curves

The approach to establish G-γ decay curves from SDMT relies on the ability of the test to provide routinely at each depth both a small strain modulus (G_0 from V_S) and a "working strain" modulus (G_{DMT} derived from M_{DMT}). These parameters could be tentatively used to develop an in-situ decay curve. In the present case, G_0 was evaluated by using V_S obtained at the same depth of retrieved triaxial samples, while G_{DMT} was derived from M_{DMT}, by using the linear elastic formula (Equation 7) suggested by Monaco et al. (2006) and Marchetti et al. (2008).

However, to use G_{DMT} it is necessary to know the correspondent elemental shear strain, here designated as γ_{DMT}. For sedimentary soils, Mayne (2001) pointed out a range for γ_{DMT} within 0.05–0.1 %, while Ishihara (2001) suggested that the range can be much larger, varying from 0.01 % to 1 %. Monaco et al. (2014) reconstructed soil stiffness decay curves for the Treporti case history from local vertical strains measured at the centre of the embankment under each load increment, concluding that γ_{DMT} was within the range 0.02–0.14 % in sand and between 0.5 % and 1.65 % in silt. Finally, Amoroso et al. (2014) in a wider study concluded that γ_{DMT} could vary from 0.015 % to 0.30 % in sands and from 0.23 % to 1.75 % in silts and clays, whereas in soft clays γ_{DMT} is higher than 2 %.

To assess the in-situ stiffness decay, Amoroso et al. (2014) suggested the following procedure:

a) Using SDMT data obtained at the same depth of each available reference stiffness decay curve, a working strain modulus, G_{DMT}, is derived from M_{DMT} and normalized by the correspondent small strain value, G_0, derived from V_S;
b) The G_{DMT}/G_0 horizontal ordinate line is superimposed to the same depth experimental stiffness decay curve, in such a way that the data point ordinate matches the reference stiffness decay curve;
c) The "intersection" of the G_{DMT}/G_0 horizontal ordinate line with the stiffness decay curve provides a shear strain value referred here as γ_{DMT}.

This methodology was applied in this study to test its efficiency in the case of residual soils. Table 3 presents the summary of the obtained results.

Table 3. Summary of CID triaxial and SDMT tests carry out in residual granitic soils for the same level of stress

Type of sample	Artificial			Natural		
	Rem 0	Mix 1	Mix 2	BH1	BH2	BH2
Depth (m)	1.04	1.02	1.02	1	1	3.4
SDMT (n.º)	CCC	CCC	CCC	1	6	6
γ (kN/m^3)$_{SDMT}$	15.70	14.86	14.97	20.6	19.1	20.8
V_s (m/s)*	230	260	362	256	236	257
G_{0_SDMT} (MPa)	84.6	102.4	200.0	137.6	108.6	140.4
$G_{max\_TRX CID}$ (MPa)	45.9	95.7	214.1	101.1	99.5	30.2
M_{DMT} (MPa)	10.0	25.3	76.1	243.1	215.8	341.3
G_{DMT} (MPa)	2.85	7.22	21.75	69.45	61.65	97.50
γ_{DMT} (%)	1.200	0.300	0.018	0.0027	0.0089	0.0007
G_{DMT}/G_0	0.034	0.070	0.109	0.504	0.568	0.695
$E_{50\%\_TRX}$ (MPa)	3.0	24.3	35.2	9.2	12.3	8.4

(*) V_s obtained from seismic refraction in artificial samples and from SDMT in natural soils.

In natural samples (Figure 6), the maximum stiffness obtained at small strain in triaxial tests converge to the same order of magnitude of the one obtained by shear wave velocities, which demonstrate the good quality achieved in the sampling processes.

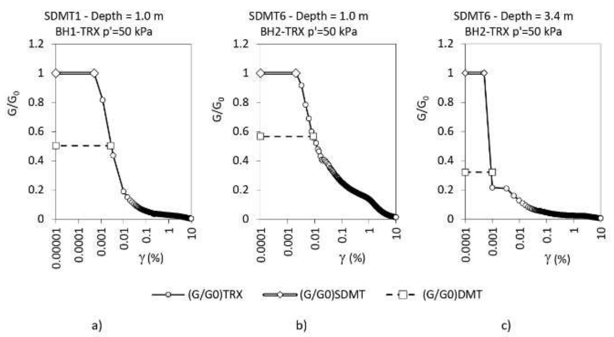

Figure 6. Laboratory G/G_0-γ curves obtained from CID triaxial tests: a) BH_1-$SDMT_1$, depth=1 m; b) BH_2-$SDMT_6$, depth=1.0 m; c) BH_2-$SDMT_6$, depth=3.4 m.

Corresponding γ_{DMT} falls within 0.0023 % and 0.0089 %, one order magnitude lower than those proposed by Amoroso et al. (2014) for sedimentary soils with similar grain size ($\gamma_{DMT}\approx$0.015-0.30 for sandy soils).

In its turn, triaxial data of artificial specimens (Figure 7) revealed that the increase in cementation level is followed by an increase in the "working strain modulus" (G_{DMT}) and decrease in the strain level (γ_{DMT}).

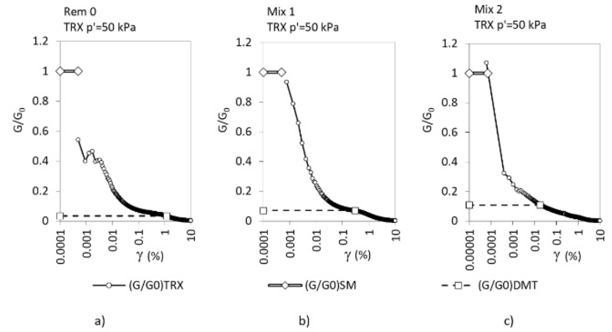

Figure 7. Laboratory G/G_0-γ curves obtained from CID triaxial tests: a) Rem 0; b) Mix 1; c) Mix2.

Furthermore, if equivalent tensile strengths in naturally and artificially cemented samples are considered (10 to 15 kPa, in the present case), it is possible to observe that behaviour are quite different, with a higher "working strain modulus" (G_{DMT}) in natural samples (Figure 8).

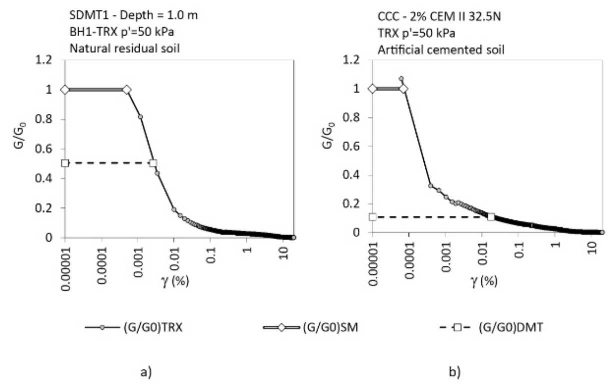

Figure 8. Laboratory G/G_0-γ curves obtained from CID triaxial tests (TRX): a) Natural sample BH1 and SDMT 1-Depth=1.0 m; b) Artificially cemented sample CCC in Mix 2.

The results arising from this framework have important consequences in the selection of working modulus for design purposes, which can be summarized as presented below.

The first conclusion is that it is possible to observe that behaviour of natural soils and artificial mixtures (remoulding the same natural material with same void ratios, same maximum deviatoric stresses and same tensile strengths) are quite different, with a higher "working strain modulus" (G_{DMT}) in natural samples, which are probably related with the different cement agents and the different fabric observed in natural samples and artificial mixtures. In fact, the existing structure in natural samples is related with bonds between grains (cementation) and fabric inherited from original rock, which are both destroyed within the remoulding process necessary to the development of artificial mixtures. Figure 9 illustrates well the differences observed in the fabric of both materials. Furthermore, the cement agent in the case of natural soils is related with bonds created between grains during the genesis of the original rock, which is then soften by weathering processes, while the agent in the artificial mixtures is a calcium carbonate industrial cement, which is evenly distributed throughout the sample.

a) b)

Figure 9. a) Natural soil; b) Artificially cemented soil.

The second important conclusion is that the higher the level of cementation of the material, the higher the value of the "working strain modulus - G_{DMT}" evaluated by the DMT test, which was expected and confirmed in the significant number of triaxial tests performed within the global research program. In fact, it can be observed that the same behaviour is observed in the triaxial tests; if, for example, we consider the secant deformability modulus at 50% of the value of q_{max} ($E_{50\%}$) represented in Table 3, the same type of evolution takes place.

The third conclusion arising from experimental results is that the higher the level of cementation of the material tested, the lower the level of strain corresponding to the mobilization of the "working strain modulus - G_{DMT}", which may be related with the fact of the strain-controlled character of the DMT tests

As consequence of these determinations, it becomes clear that the evaluation of M_{DMT} (and obviously, E_{DMT} and G_{DMT}) is always related to constant membrane displacements of 0.05 and 1.10 mm. The strain on the displacement direction (ε_L) is defined as:

$$\varepsilon_L = \frac{\Delta L}{L} \; ; (\%) \tag{9}$$

where ΔL stands for the displacements of the membrane (0.05 or 1.10 mm) and L is the extension of the massif affected by loading,

Once the test displacements are constant and ΔL is also constant, then the decrease of strain (ε) or distortion (γ) can only be explained with the increase of stress transfer field.

6. Conclusions

The analysis of experimental data arising from the specific framework related with deformability of granitic residual soils from Guarda have shown the following important trends:

a) The higher the level of cementation of the material, the higher the value of the "working strain modulus - G_{DMT}" evaluated by the DMT test.
b) The higher the level of cementation of the material tested, the lower the level of strain corresponding to the mobilization of the "working strain modulus - G_{DMT}".
c) Stiffness behaviour related with natural and artificial samples with similar void ratios, maximum deviatoric stresses and tensile strengths evaluated by DMT test are clearly different, showing a much higher "working strain modulus" (G_{DMT}) in natural samples.
d) Triaxial data show the same trends referred above, validating the response of the soil when tested by DMT.
e) It is most probable that the trends observed in these two tests will be followed in other in-situ tests.

Taking these findings into account, the adoption of stiffness parameters for design purposes must be done with care, to ensure representative calculations of the expected settlements in each specific situation. This will depend greatly in the gap between the strain levels involved in the test measurement and those expected in the real work situation. Being so, direct application in design calculations of the characteristic stiffness derived from DMT tests may lead to conservative or unconservative results, depending on the relation between test and work strains. DMT results will be unconservative when the working strains are higher than DMT's and conservative in the other way around.

As consequence, it becomes fundamental to select the stiffness parameters for design from adequate stiffness decay curves, which can be developed from triaxial tests or by test results corresponding to different levels of strain, such as the logistic curve proposed by Rodrigues et al. (2020) for the case of the residual soils of Guarda tested by SDMT tests.

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