Characterisation and Engineering Properties of Natural Soils – Tan et al. (eds.)
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Characterising and deriving engineering properties of a saprolitic soil from granite, inn Porto

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ABSTRACT: Residual soils are characterised by the presence of a bonded structure and fabric that has significant influence on engineering behaviour. This characteristic requires specific concepts for geotechnical classification. Micro and macrofabric analysis can easily clarify the potential for more or less pronounced metastability. Deformational behaviour, pre-yield and post yield, is related to fairly well defined state regions. A good definition of constitutive laws of these soils demands careful experimental work and parametric analysis both from *in situ* and laboratory testing. A typical profile of a saprolitic soil from granite in Porto was studied. Several *in situ* testing techniques (CH, SPT, CPT, PMT, SBPT, DMT, PLT and a full-scale footing testing) were used in order to have a very sound knowledge of the mechanical characteristics of this soil. High quality undisturbed block samples were recovered and an extensive laboratory testing programme was carried out, including isotropic and oedometric consolidation tests, as well as CID and CK_0D triaxial tests using local strain measuring devices. This paper presents and discusses the results obtained, giving emphasis to the interpretation of constitutive behaviour differences in stress–strain levels lower and higher than the yield stresses related to saprolitic soils with weak relic structures.

1 INTRODUCTION

Residual soils are widely distributed in tropical regions, with frequent and deep horizons. This is associated with climatic conditions where high temperatures and precipitation dominate, the former factor being favourable to the development of chemical reactions largely responsible for rock weathering. The abundance of water, particularly the generation of underground flows, is a benchmark for leaching processes and the inherent creation of new structural (micro and macro) matrices and, even, new fabrics.

In the North-Western Region of Portugal residual soils from granite are dominant (Viana da Fonseca et al., 1994). Here the climate is temperate and geologically granite rocks prevail. Due to their specific genesis such soils present complex characteristics, which are a consequence, on the one hand, of their overall variability and inhomogeneity and, on the other hand, of the spatial arrangement and distribution of the particles and pore spaces.

There are several Civil Engineering works, where the costs, times, or construction processes, are strongly conditioned by the quality of the geotechnical characterisation of these formations. This applies to several areas of intervention: foundations, excavations and supporting structures, tunnelling, etc. where most difficulties, drawbacks or, even, accidents are associated with insufficient knowledge of specific indexing criteria and geomechanical characterisation frameworks, somehow distinct from classical proposals.

As will be emphasised in the present paper, it is assumed that residuals soils are characterised by the presence of a bonded structure and fabric, having significant influence on engineering behaviour and requiring specific concepts for geotechnical classification. Micro and macrofabric analysis can clarify the potential for more or less pronounced metastability (Vaughan, 1988).

2 ENGINEERING GEOLOGY

2.1 *Source of material*

The typical Porto granite is a leucocratic alkaline rock, with two micas and of medium to coarse grain size. The chemical and mineralogical constitution of this rock varies naturally, generating a fairly

Figure 1. Illustration of a weathering profile – Northern Portugal.

heterogeneous mass. In what follows, reference will be made to a specific site and profile where some chemical and mineralogical analysis were conducted on both sound and weathered rock (see following paragraph).

2.2 *Formation process: weathering*

The main weathering factors acting on granite masses seem to be the discontinuity degree – that is, number, spacing, orientation and continuity of joints – and the proximity of different tectonic events that determine the creation of weak zones where the water actions tends to increase by flow on discontinuities. Depth of weathering ranges typically from 0 to 20 m with more common values of 5 to 9 m (Figure 1).

Weathering factors and processes relevant to residual soils are the following: (i) localization; (ii) topography (particularly the natural surface slopes); (iii) mass jointing and composition of the parent rock; (iv) climate (actual and past) – rainfall intensity, temperature gradients, humidity; (v) hydrology (water levels, seasonal gradients, percolation); (vi) vegetation.

Of these factors, special reference should be made to climatic conditions. In fact, while in tropical areas the high values and gradients of temperature are responsible for acceleration of chemical and mineralogical processes, in temperate zones, rainfall and underground and sub-superficial water conditions will have a major influence on them. Hydrolysis and liberation of dissolved mineral constituents will carry kaolinisation and sericitisation of the feldspars and some chloritisation of iron-magnesium minerals. Thin to very deep weathered profiles (as much as up to 30 m) are common in Portugal mainly in large areas in the littoral, where these climatic conditions and the factors listed below co-exist. In opposition to tropical conditions, re-cementation by concentration of sesquioxides and recrystallization processes are very rare, as consequence of the lack of thermal factors, and laterites or mature residual soil will not be found.

Also important is the parent rock composition, fabric and jointing. Although recognising some variation, the most weathered granite masses are usually those that: (i) present medium to large grain size; (ii) possess large amounts of sodic feldspars (such as plagioclases) and to some extent potassic feldspars (like orthoclase); and, (iii) contain discontinuities and fractures, as well as some intrusion of other masses such as pegmatites and migmatites.

2.3 *Typical profiles*

Specific geomorphologic and topographic conditions are also determinant, the deepest weathered profiles being associated with rather incidental relief with accentuated cuttings, possibly with discontinuities, creating subsequently fairly good drainage conditions (external and internal). In littoral zones, particularly in Porto, ancient tectonic movements, as well as global hydrothermal movements, have been associated with marine transgression (detected by some reminiscent signs) leaving deep altered profiles, by means of strong chemical processes.

Weathering processes take place in a large scale (Viana da Fonseca, 1996). Coarse and resistant quartz grains are bonded by fragile clayey plagioclase bridges and result in soils with medium to high porosity fabric. The feldspars are subjected to intense weathering processes, typical of high average annual precipitation and well-draining ground profiles.

The occurrence of a weathered superficial zone with residual soils whose thickness in some cases reaches 20 m or more is typical in the coastal zone where most urban centres – as the Porto urban area – are concentrated. A sound geotechnical knowledge of these soils is therefore of great interest. The profiles often have a very erratic evolution of the weathered masses (classes) in depth and are usually characterised by unpredictable variations in depth. Some relation is due to the discontinuities, as referred above, and very thoroughly associated to water levels and flows.

The most intensely weathered zones are the classes of W_5 and W_6 (ISSRM, 1981, IAEG, 1981) that may be described as follows:

- W_5: the rock is completely weathered – decomposed and/or disintegrated to soil. The original mass structure is still largely intact. These soils are usually designated as "saprolitic soils" or "young residual soils".
- W_6: all rock is converted to soil. The original mass structure and material fabric have been destroyed. There may be a significant change in volume, but the soil has not been transported. These soils are associated with granular matrices with no leaching and structuration and in tropical regions turn into "lateritic soils" or "mature residual soils", by secondary processes of re-cementation.

Most of the ground masses that involve geotechnical works in the Porto region, and concern geotechnical designers, are included in the W_5 group. They can stand over significant depths from the surface. These saprolitic horizons, because of their relevance, will be the main subject of this paper.

2.4 *Preliminary classification of the saprolitic horizon*

Characterisation of saprolitic soils demands other criteria than those usually considered in classical Soil Mechanics classifications. Identifying them as "hard soils" or "weak rocks" is not sufficient to define their mechanical behaviour. A more comprehensive classification should include a description of the weathering profile as well as significant information on chemical, mineralogical and physical aspects of their constituents. Also, preliminary indications of the duality "strength-consistency", by simple parameters (such as unconfined strength), can constitute fair steps for a more complete description of these materials.

Parameters that result from identification tests on remoulded soil, such as Atterberg limits or grain size distribution, do not reveal or classify, in contrast to transported soils, the geotechnical behaviour of these soils (Vaughan 1988). Remoulding and preparing these masses for analysis change their characteristics, especially for natural state responses and jeopardise the use of laboratory processes for characterisation of such materials that are strongly controlled by structure and fabric.

Novais Ferreira (1985) indicates four aspects that should be included in the classification of these soils: morphological and mineralogical characteristics, physical identification testing and engineering properties. The author presents the following expeditious parametric evaluations: hygrometrical state, colour (or discoloration), strength-consistency (simple methodologies such as penetration of the geological hammer, remoulding between fingers, disaggregation in "slake durability tests", or, even, uniaxial compression strength), fabric (taken as the macrostructure, particle and voids size, arrangement and distribution, as well as fissuring), texture (particle sizes and shapes), density, apparent mechanical and hydraulic properties, mineralogy (especially the formation of minerals due to weathering processes).

2.5 *Chemical indices*

During the weathering process there is a differential evolution of chemical constituents (related to mineral evolution). Therefore, there is a tendency to adopt some reference elements that are supposed to be unaltered during that evolution.

Rocha Filho et al. (1985) have discussed some of the chemical indices used in the literature for this purpose, and concluded that the chemical indices based upon the total silica quantity as a means to evaluate weathering degrees of acid rock (as granites) can be misleading. The reason is that silica, during weathering, can concentrate in the form of quartz, leading to an increase in these indices, while the median values must solely be related to the original material. Simple ratios between the most stable independent constituents in weathered and sound rock conditions, such as Al_2O_3, can be used as a reference for the analysis of weathering degree. This has been done with good results, but other analyses have demonstrated that in weathering there were:

- small losses of silica (SiO_2) and, above all, of potassic elements (K_2O), as well as some iron oxides (Fe_2O_3);

– large losses of solid and calcareous elements (Na_2O, CaO) as well as other iron oxides (FeO) and magnesium (MgO);
– substantial gains in water or ionised water (H_2O, H_2O^+).

These latter elements, particularly the H_2O^+, are presented as possible indices of weathering profiles (Jayawardena, 1993).

2.6 *Mineralogical indices*

The points referred to above in relation to a synthesis of chemical evolution have consequences in the mineralogical evolution of these rocks due to weathering, as can be seen from the following analysis of the same samples that have been chemically analysed.

Viana da Fonseca (1996) studied in some detail the evolution pattern of mineralogical constituents of the Porto granite. Some of his conclusions areas follows:

– quartz is naturally very stable and its stability can even be increased by silica concentration;
– potassic feldspar (orthoclase) evolves to kaolinite and, to a lesser extent, to muscovite this this behaviour is directly related to small losses of K_2O;
– sodic feldspars (albite) – quite important in this alkaline type of granite – and calcareous feldspars (anortite) – these present only in small quantities – are extensively transformed into kaolin and muscovite.

The relation of all these transient constituents to the stable amount of quartz (even considering some variation of this component, quartz is very stable during the weathering process) is usually used as a classification weathering index, but other primary elements such as zircon, tourmaline, can also be used.

Lumb (1962) has defined the degree of decomposition for granitic rock as a function of the quartz and feldspar existing in both parent rock and residual soil:

$$X_d = (1 - W_f/W_{f0})/(1 + W_f/W_q) \tag{1}$$

where W_q is the weight of quartz, "unaltered" by weathering, and W_f and W_{f0} are the weights of feldspar in the soil and in the parent rock, respectively. The lixiviation index is also used and was defined as (Rocha Filho et al. 1985):

$$\beta = ba_1 \text{ (weathered rock)}/ba_1 \text{ (sound rock)} \tag{2}$$

where

$$ba_1 = (K_2O + Na_2O)/Al_2O_3 \tag{3}$$

The values obtained for these indices for Porto granite ranged between 0.59 and 0.63 for the Xd, and 0.40 to 0.50, for β, reflecting clearly a high degree of weathering for this soil. The range for β is very similar to the range reported by Rocha Filho et al. (1985) for Brazilian saprolitic soils from granite ($\beta = 0.36$–0.50).

It is interesting to observe that fine fractions ($< 2\mu m$) contain the highest percentage of kaolinite, which is abundant in the most superficial horizons, in contrast to gybsite which is rare in these levels and most intense at depth. X_d values correspond, in Collins (1985) chart, to a "metastable" zone, associated with a porous but cemented interparticle matrix (see Figure 2). This means that this saprolitic soil is composed of coarse quartz grains bonded by a porous clay matrix resulting from the chemical alteration of the plagioclases. These feldspars lead to important lixiviation processes.

2.7 *Void ratio and dry density*

Void ratio and dry density have already been referred to as particularly inadequate for predicting mechanical behaviour. In these saprolitic soils these physical parameters usually take the following values for dry apparent densities ($d = \gamma_d/\gamma_w$):

Sound granites (W_1):	2.55–2.70
Medium to compact weathered altered granites (W_2–W_4):	2.30–2.60
Saprolitic soils (decomposed granites) (W_5):	1.45–1.77

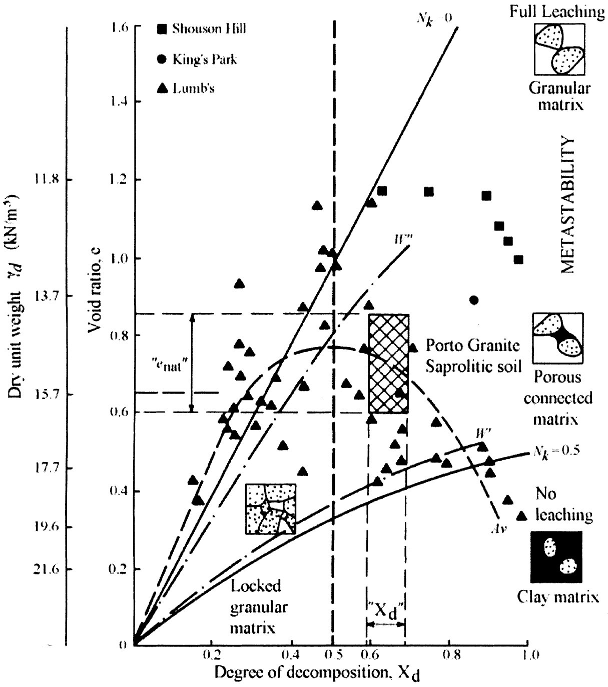


Figure 2. Microstructure characterisation (Collins, 1985).

Table 1. Trend values of uniaxial compression strength for weathering classes.

Weathering degree	W_2–W_3 W_3–W_2	W_3	W_4–W_3 W_3–W_4	W_4	W_4–W_5	W_5
σ_u (MPa)	≈10	≈35	≈20	≈15	≈10	0.03–0.14

Table 2. Common natural physical parameters.

γ_s (kN/m^3)	w(%)	γ (kN/m^3)	Sr (%)	e	k(m/s)
25.7–26.5	15–25	15.0–18.5	80–100	0.40–0.70	10^{-6}–10^{-5}

It is noticeable that there is a smooth decrease of apparent densities between sound rock and the overlying weathering horizons – still rock masses but with clear signs of alteration. A steep drop in density is usually observed when passing to the highest weathering degrees (saprolitic and residual soils), even though preserving macro structures and fabrics from the parent rock.

It is obviously very important to note that the distinction between the intermediate weathering degrees is made by means of other specific indices, which are more useful and objective in Rock Mechanics Classification for geotechnical design, such as RMR (Bieniawski, 1976) or Q (Barton et al., 1974) Classification. On these different levels a marked distinction is due to uniaxial compressive strengths and characteristics of joints, defining each geomechanical unit. Although this is not the aim of this paper, which will solely focuses on the characteristics of W_5 horizon, an illustration of these patterns (ranging from W_1 to W_5) is presented in Table 1.

Table 2 presents typical regional values of physical parameters of W5 horizons ("saprolitic soils"). The relatively low values of total unit weight (typically between 17 to 19 kN/m^3) are associated with a flocculated structure (open continuous voids in a cemented, bonded, structure).

In order to have a more objective characterization of a specific profile, a detailed description of an experimental site where a significant saprolitic horizon dominated will follow.

2.8 *Permeability*

The importance of a good evaluation of the hydraulic conductivity, is related to problems of slope stability and flow through dam foundations as well as abutments, deep excavations and retaining structures. Usually it is the macro-structural variability – mainly in saprolitic soils – that determines the in situ behaviour. It is, then, very difficult to evaluate its real patterns by laboratory testing, even with good undisturbed samples. Here the scale effects are of main concern.

Costa Filho & Vargas Jr. (1985) synthesize several works (some from the Geotechnical Control Office – GCO, 1990) proving that the influence of relict structures from the parent rock (veins-impermeable and permeable-fissures or fractures, discontinuities, etc.) dominate in determining the water flow conditions in granite weathering profiles.

Although these are very important in younger residual soils, as saprolitic soils, factors like macropores and open channels in cemented and flocculated matrices appear to be important (mainly in laterites). These authors state as less important the microstructural arrangement on permeability characteristics. In a simple general model Deere & Patton (1971) have suggested a classification of this characteristic with weathering degrees (Table 3).

The horizons defined as saprolites (II-A) or saprolitic soils – young residual soils – (I-C) are usually medium to high permeability materials. In the latter this characteristic is due to the presence of sandy and silty grains aggregated in large pore sizes (flocculated matrices) and in the former (weathered rock masses) to the highly fractured matrix. Variations of "k" with depth are small and are related to stable open microstructures and homogeneous dispersion of discontinuities.

There has been a very significant recent investment in experimental campaigns in the urban area of Porto for the new subway project, in particular for the evaluation of permeability variation with depth, and its relation to weathering degrees, as this information is important for the calculation of pumping volumes for the design of deep excavations and retaining structures in the future stations. Although it is rather difficult to generalize, Table 4 indicates some of the trends that have been detected in these studies.

(1) qualitative trends as revealed in Deere & Patton (1971), Dearman (1976), Costa-Filho & Vargas Jr. (1985).
(2) experimental values obtained in variable head tests through sections in drill holes or pump tests, with surrounding piezometers.
(3) More kaolinitic matrices present values of permeability that are lower than the indicated ranges by one order of magnitude.

Table 3. Description of permeability of a weathering profile in igneous and metamorphic rocks (Deere and Patton, 1971).

Zone		Relative
Costa Filho & Vargas Jr. (1985)	Deere & Patton (1971)	Permeability
IA-I horizon	Organic soil	Medium to high
IB-B horizon (W_6)	Mature residual soil and/or colluvial	Low (generally medium or high in lateritic soils if pores or cavities present)
IC-C horizon (W_5)	Young residual or saprolitic soil	Medium
II-A transition of residual soil to weathered rock (W_4–W_2)	Weathered rock	High to medium

Table 4. Trendy values of permeability by classes of weathering of Porto granite.

	Permeability (m/s)	
Class of rock weathering (W_i from ISRM, 1981)	References (1)	Exp. results (2)
Decomposed rock – soil with no relic structure (W_6)	Low	$\approx 10^{-7}$
Completely weathered rock – saprolitic soil (W_5)	Medium	10^{-6}–10^{-5} (3)
Highly weathered (W_4) and fractured (F_4–F_5) rock	High to medium	10^{-5}–10^{-4} (3)
Moderately weathered (W_3) and fractured (F_3–F_4) rock	Medium to high	10^{-5}–10^{-6}
Slightly weathered rock (W_2)	Medium	10^{-6}–10^{-7}

Several in situ tests for determination of permeability are presented in the "*State-of-the-Art*" report from Costa-Filho & Vargas Jr. (1985), involving borehole tests with and without packers, exploratory pits and trenches, using constant and variable heads.

These borehole tests are the most appropriate methods, as they reflect macro and microstructural effects on this behaviour, the following trends, for saturated conditions, can be summarised:

– presence of fissures and cracks, natural or induced by earthquakes, are extremely important in the resulting overall high values of permeability;
– in the saprolitic soil horizons, those macrostructural features are less pertinent and this results in fairly uniform values of permeability with depth;
– in granite masses these permeability values are relatively isotropic, resulting from an isotropic matrix with almost constant porous arrangement with depth;
– the use of pits and boreholes in the determination of permeability can overpredict permeability due to reduction of the effective stresses by swelling;
– in saprolitic soils from granite the differences between laboratory and field test results are low due to the high spacing of macrostructure effects but the evaluation of representativeness of sampling and testing must be dealt with

3 DESCRIPTION OF THE EXPERIMENTAL PROFILE

3.1 *Experimental program*

A detailed experimental work was carried out at a given site in the region of Porto, of which a fairly homogeneous saprolitic soil 6 m thick was encountered. The central element of the experiments was the execution of a load test on a 1.20 m diameter rigid reinforced concrete footing. In association with this test, an extensive geological and geotechnical characterisation was undertaken, including *in situ* testing (CH – cross-hole, SPT, CPT, PMT, SBPT, DMT, PLT) as well as laboratory testing for chemical, mineralogical and physical identification, and oedometer and triaxial testing on high quality block samples.

3.2 *Physical identification*

Evaluations of particle size distribution, Atterberg limits and abrasion indices ("Los Angeles" or "Slake Durability" tests) indicate that the composition and texture of the parent rock do not have significant influence on the results. Saprolitic soils originated from fine, medium or coarse grained granites present always well graded particle size distribution and low plasticity indices, and are systematically classified as silty sands (SM) or, rarely, as clayey sands (SC).

Figure 3 shows the particle size distribution and the associated plasticity and activity charts proposed by Vargas (1992) – based on Atterberg limits and clay percentages – from determinations on 22 SPT samples and 10 block samples. Almost 30% of these evaluations resulted in Non-Plastic soils, and are not presented in the figure.

Fines are classified as ML, revealing low plasticity. Typical values of w_L up to 40% and I_P up to 13% reflect the influence of the high percentage of laminated particles (mica and feldspar) on retaining the water. This water is not, however, a real sign of the true plastic behaviour of the overall soil. Physical parameters evaluated over a substantial number of specimens taken from undisturbed block samples and their comparison with typical regional values (Viana da Fonseca et al., 1994) revealed excellent convergence. It should be added that the most frequent saturation degrees are around 90–95%, which correspond to very low suction levels. The position of the water level was very stable during the experimental activities, around 1m below ground surface.

3.3 *Permeability*

To examine the influence of microstructure a simple study of permeability variation with confinement stress has been made. Triaxial samples were subjected to a constant "in-flow" pressure and confined with increasing stresses. Two zones with some transition were observed, reflecting an initial relatively stable open structure (the stability is due to loading between coarser grains) that evolves to a more classical sedimentary behaviour for high stresses, after de-structuring. Values of permeability are as follows:

$$k = 200\, D_{10}\, e^2 = 24{,}91 \times 10^{-5} \times e^2 \rightarrow D_{10} = 0{,}0011\,\text{cm}$$
$$k = 200\, D_{10}\, e^2 = 40{,}83 \times 10^{-5} \times e^2 \rightarrow D_{10} = 0{,}0014\,\text{cm}$$

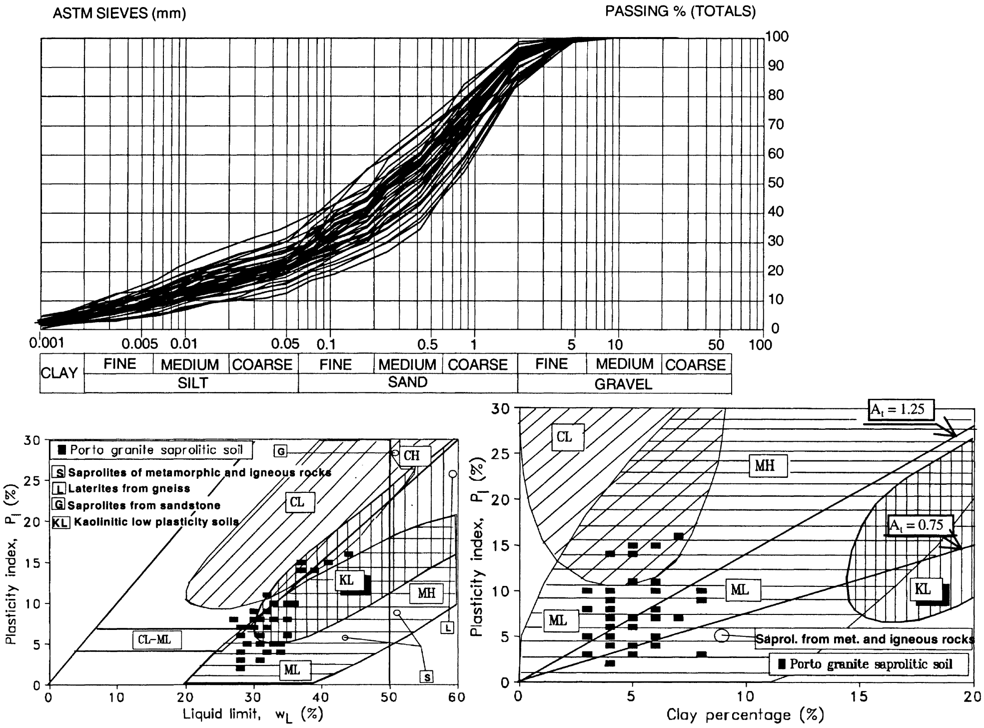


Figure 3. Particle size distribution and plasticity and activity charts (Vargas, 1992).

This latter is closer to real values of D_{10} than the former, which is underevaluated because of the secondary effect of "bonding".

Permeability determinations for undisturbed and remoulded (to the same physical conditions – γ and w) samples revealed values of k dropping by factors of 10 to 100:

$$k \text{ (und.)} \cong 3\text{–}4 \times 10^{-5} \text{ m/s (as for ranges in } W_5\text{: } k = 10^{-6}\text{–}10^{-5} \text{ m/s)}$$

$$k \text{ (rem.)} \cong 7\text{–}8 \times 10^{-8} \text{ m/s (as for ranges in } W_6\text{: } k = 10^{-8}\text{–}10^{-7} \text{ m/s)}$$

3.4 *State index*

As structured materials, residual soils are difficult to characterise from a geotechnical point of view (Clayton & Serratrice, 1997). Particle size distributions and Atterberg limits (deeply influenced by the inclusion or not of the coarser grains or the different moisture testing conditions) are strongly irreproducible. Vaughan (1988) suggested the use of the "relative void ratio" defined by:

$$e_r = \frac{e_0 - e_{opt}}{e_L - e_{opt}} = 1 - D_r \tag{4}$$

e_0 being the natural (*in situ*) void ratio (in this case: $e_0 \cong 0.7$), e_{opt} the void ratio corresponding to Proctor optimum moisture condition and e_L the void ratio at the liquid limit, determined by the cone penetration method (BS 1377) taking all soil (including that retained on sieve n. 40 of ASTM standards).

From several determinations on representative samples, the following average results were obtained: $w_L = 29\%$; $e_L = 0.793$, corresponding to indices: $e_r = 0.72$ and $D_r = 0.28$.

D_r takes a low value, typical of loose granular soils, but does not reflect the mechanical behaviour patterns observed in these soils, as will be discussed below.

4 CHARACTERISATION OF THE SAPROLITIC SOIL BY *IN SITU* TESTS

4.1 *Soil profile identification*

The experimental work was carried out at a site (approximately $50 \times 30\,m^2$) in which a homogeneous saprolitic soil 6 m thick was identified by a previous SPT and DP (DPSH and DPL) exploring campaign. Geologically, the parent rock is representative of the granite from Porto region, described above. The photo included in Figure 4, taken at the end of the experimental campaign and prior to the construction of a new building for a District Hospital, gives a clear overview of the general saprolitic profile developing with depth.

Apart from the natural spatial variation of the structure and fabric of these residual soils due to some preserved relic heritage, there is evidence of a fairly homogeneous pattern of ground profile in terms of a geotechnical view. As referred to above, results obtained with specimens taken from the SPT sampler and from blocks revealed a fairly homogeneous ground profile.

4.2 *Summary of the testing program*

Table 5 summarises the tests performed and Figure 5 presents the plan of the experimental site.

4.3 *General trends from CH, SPT, CPT and DMT*

The results of SPT, CPT and DMT tests and the values of the maximum shear modulus, G_0, obtained from CH tests are shown in Figure 6. It is observed that the CPT cone resistance, q_c, the N_{60} from SPT

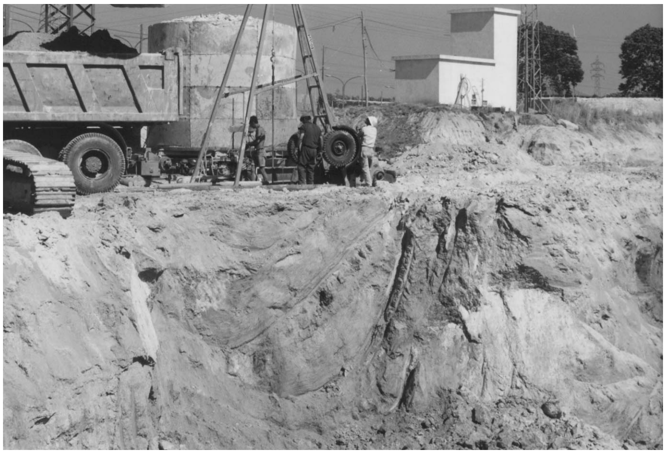


Figure 4. Overview of the weathered profile revealed in a fresh cutting on the experimental site.

Table 5. In situ tests.

Test	Number (penetration – m, boreholes – bh)
SPT	46 (4 bh)
DPSH	3 (33.0 m)
DPL	15 (66.3 m)
CPT	9 (53.9 m)
PLT (D = 0.3 m)	3
PLT (D = 0.6 m)	3
Footing (D = 1.2 m)	1
PMT	5 (3 bh)
SBPT	4 (2 bh)
CH	32 (3 bh)
DMT	12 (1 bh)

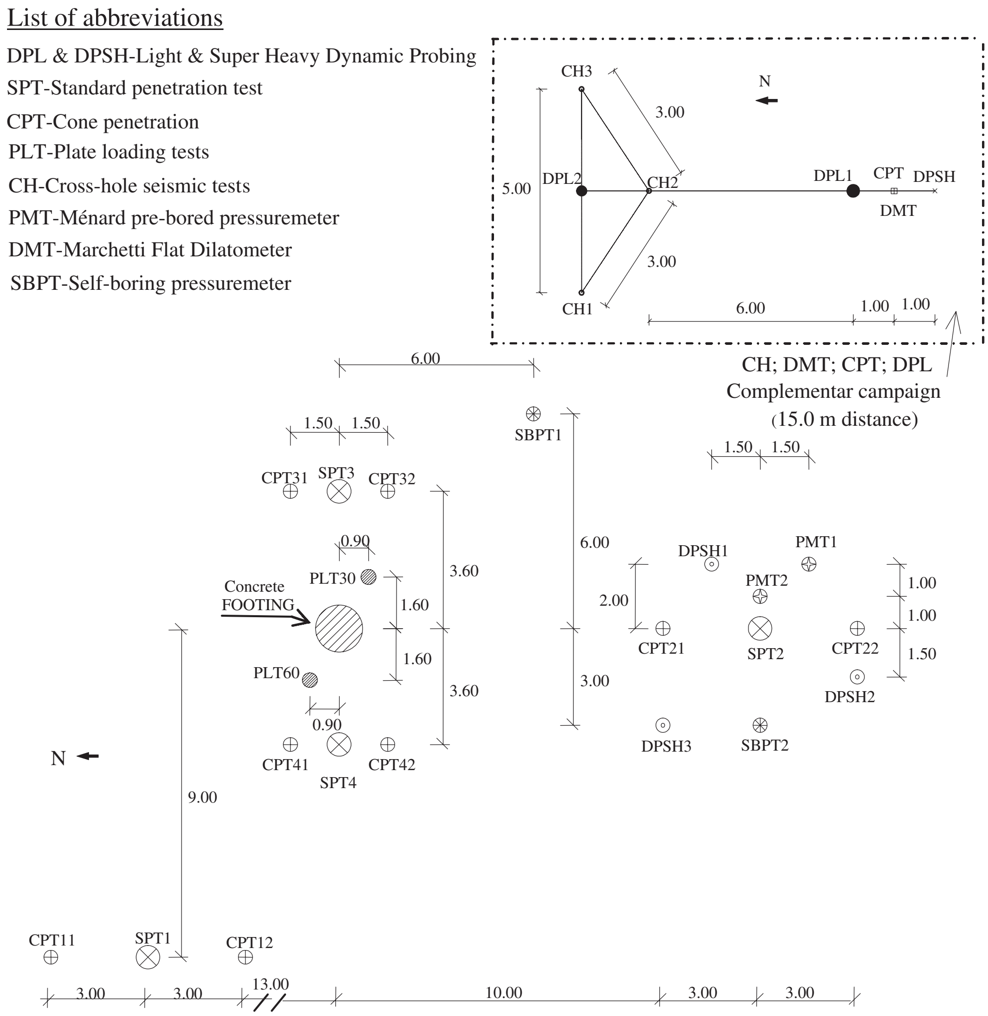


Figure 5. Plan of the experimental site.

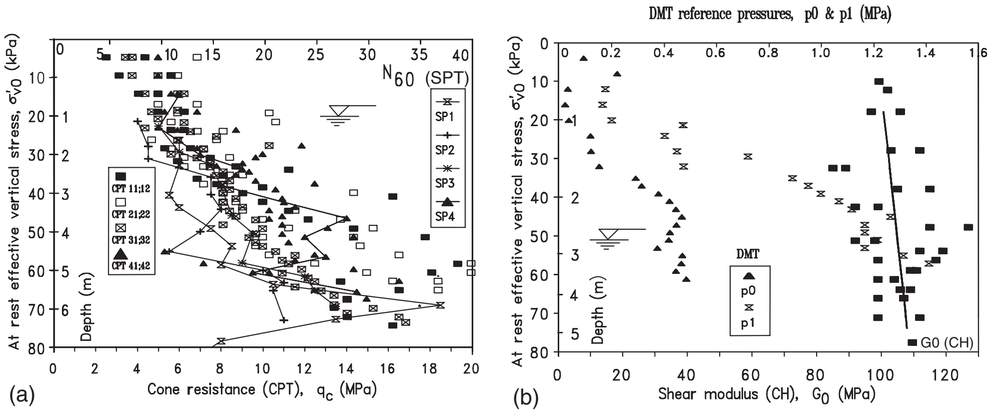


Figure 6. *In situ* test results: (a) SPT and CPT; (b) CH and DMT.

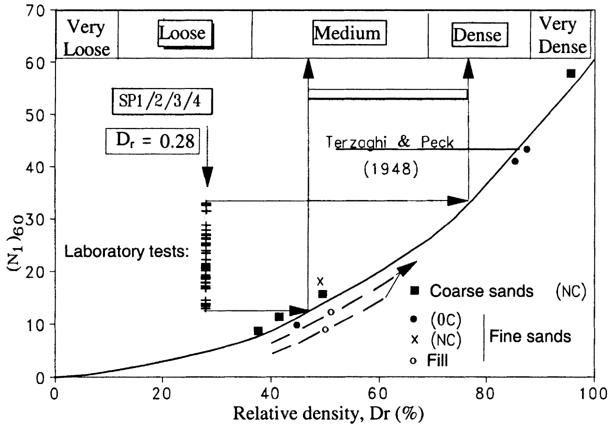


Figure 7. $(N_1)_{60}$ versus D_r (Skempton, 1986).

and p_0 and p_1 from DMT, increase noticeably linearly with depth (or the at rest effective vertical stress, σ'_{v0}), whilst G_0 reveals a very low variation.

4.4 *Standard penetration testing (SPT)*

The proposal of Décourt (1989), based on the variation of $(N_1)_{60}$, was used to derive a range of values for the angle of shearing resistance of 36°–40°, with an average value of 38°. These values agree with those deduced from back-analysis of PLT and triaxial tests on undisturbed samples (see description below of lab results). Nevertheless, the cohesive intercept calculated from these latter tests is significant ($c' = 8$–12 kPa), which is a characteristic of residual soils and, therefore, strength solely defined by ϕ', as derived from SPT tests, will be rather conservative.

Values of $(N_1)_{60}$ plotted on Skempton's (1986) graph (Figure 7) would indicate medium to high densities while direct evaluation of relative density ($D_r \cong 0.3$) indicates high porosity. It is believed that analysis ruled by dilatant theories will be conservative. Besides, the assumption of OCR values greater than one for mechanical characterization of these soils by means of *in situ* tests such as SPT, deviates the results towards abnormal low strength values.

For the variation of N_{SPT} values with stiffness modulus it is well stated that correlations between penetration parameters and maximum shear modulus (G_0) are the ones that best assure some independence on unavoidable misleading factors, such as scale effects, non-linearity, etc (Jamiolkowski *et al.*, 1988; Robertson, 1991). From the experimental data a linear relation could be obtained:

$$G_0\,(\text{Mpa}) = 98 + 0.42 \cdot N_{60} \tag{5}$$

The variation of G_0 with effective mean stress (σ'_{m0}) is significantly less accentuated than the corresponding penetration parameters, such as N_{60}. As a consequence, correlations between G_0 and N_{SPT} for the relevant values of σ'_{m0} for shallow foundations strongly underestimate elastic stiffness of the soil (Stroud, 1988).

This subject of simultaneously using such two tests for the definition of the constitutive law of soils, has been recognised as very promising, as one – the Cross-Hole – applies to very low strain levels while the other – the SPT – induces large strain (beyond failure) giving rise to some confidence in the evaluation of stiffness variation with strain (or stress). Some authors (Fahey, 1998, 2000) argued that this is more reliable if a less destructive test, instead of the "robust" SPT, is used, such as CPT or, even better, a dilatometer (DMT). This is being studied in these regional residual soils from granite. Some conclusions are expected in the future.

4.5 *Dynamic probing (DPSH and DPL)*

The two regionally most common energies from ISSMFE standards were used: super-heavy (DPSH) and light (DPL). The variation of DPSH parameters with depth is less pronounced than the N_{SPT} and q_c (CPT) variation. Even an analysis based on the specific energy of penetration (Nixon, 1988), particularly between the results of DPSH and SPT, is not conclusive. On the other hand, the results of DPL tests, executed alongside CPT and SPT tests, and interpreted in terms of the dynamic cone resistance (R_d – Dutch formula) are very similar to q_c values. This trend has been regionally verified on sandy soils and saprolitic soils from granite, strongly weathered with mainly sandy matrix, to moderate depths (down to 10 m from the surface). The fact that this trend is undetected in the DPSH test is associated with its high energy, leading to low number of blows, therefore insensitive to natural variation of soil characteristics with depth.

4.6 *Cone penetrometer testing (CPT)*

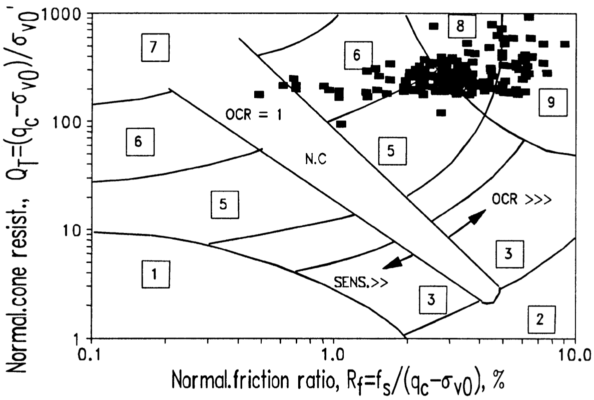
Results of CPT (Fig. 6b) show an approximately linear increase in q_c with σ'_{v0}. It is also observed that friction increases smoothly with the vertical effective stress, with ratio of $f_s/\sigma'_{v0} \cong 7$, revealing fair homogeneity of friction over this depth.

Soil classification based on K_0, OCR and f_s/σ'_{v0} values, with reference to classifications developed for transported soils, indicates high OCR values. This is not congruent with the genesis of these soils and with the low values of K_0 regionally observed (details below).

Classification by Robertson (1990) chart identifies this material as cemented, aged or very stiff natural soil, with a grain size distribution typical of sands or silt/sand mixtures, although its density index values are low (Figure 8).

Danzinger et al. (1998) collected data from tests sites of different parent rocks of residual soils, to conclude that correlations between CPT and SPT present a large scatter due to intrinsic heterogeneity. These authors have concluded that different parent rocks generally produce different correlations for the same particle size distribution. The Brazilian data shows a general trend of lower values of the relation of q_c/N_{SPT} with D_{50} than that expressed by Robertson & Campanella's (1983) average line. Our results, however show the opposite as it is illustrated in Figure 9. The results, corresponding to $q_c/N_{60} = 0.7–0.8$ (q_c, MPa), are above the mean trends of the charts, which is probably a consequence of the greater sensitivity of CPT cone resistance to the cohesive strength component of the residual soil. This trend corroborates other Brazilian author's (e.g. Rocha Filho, 1986) findings in residual soils, but demand more regional studies.

The evaluation of the angle of shearing resistance by means of Robertson & Campanella's (1983) proposal led to higher values of ϕ' (44–45°) than those obtained from triaxial tests, where the cohesive component is non-negligible. This reflects the simultaneous sensitivity of q_c towards frictional and cohesive components. Robertson & Campanella's methodology, being strictly frictional, does not explicitly derive the cohesion intercept, although the value of the angle of shearing resistance has to be taken as a secant value on the stress space and, consequently, decreases with increasing vertical effective stress, a fact that is very well defined in the results plotted in the authors graph. Nevertheless, care should be taken in the adoption of this solely frictional resistance for the evaluation of the ultimate load of shallow foundations



Zones: 5 - sandy mixtures - silty sand to sandy silt;
6 - sands - clean sand to silty sand;
8 - very stiff sand to clayey sand (cemented);
9 - very stiff, fine grained (cemented);

Figure 8. Soil classification (Robertson, 1990).

as it is well recognised that there is a strong influence of cohesion on the evaluation of q_{ult}, as well as the strong non-linearity of a bearing capacity factors with increasing values of ϕ'.

An attempt was made to configure the sleeve friction parameters by back-analysing CPT results in order to obtain the best pair of strength parameters using formulations based on Janbu & Senneset´(1974) theory. Values of c′ between 15 and 40 kPa were obtained, which clearly overestimate those obtained by back-analysing multiple plate loading tests and triaxial tests on undisturbed samples, discussed below. On the same theme, use of the Puppala et al. (1993) interpretation has resulted in a very strong over-evaluation of effective cohesion, that can be explained by the different cementation conditions between these natural formations compared with the artificially cemented sands used by the authors.

From the interpretation of the CPT results by the state parameter concept, ψ, the values taken from the normalised tip resistance have led to higher strengths than those evaluated directly from the natural void ratio; this seems to be related to the fact that the mechanical properties of these soils are not exclusively conditioned by volumetric factors but mostly by fabric and structure.

Values of q_c were correlated with G_0 and a similar low degree of dependency with penetration resistance was detected. From the integration of these values in Robertson's (1991) proposal for transported soils in calibration chamber, with different overconsolidation ratios (Figure 10), it is verified that measured

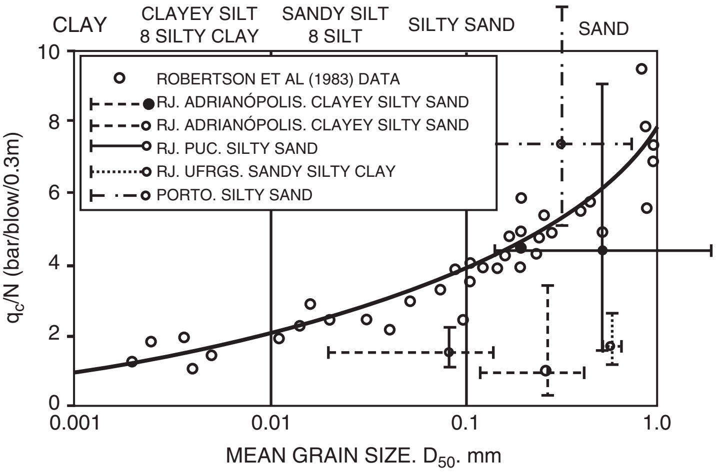


Figure 9. Ranges of the relation of qc/N versus D_{50} on residual soils (Danzinger et al.,1998).

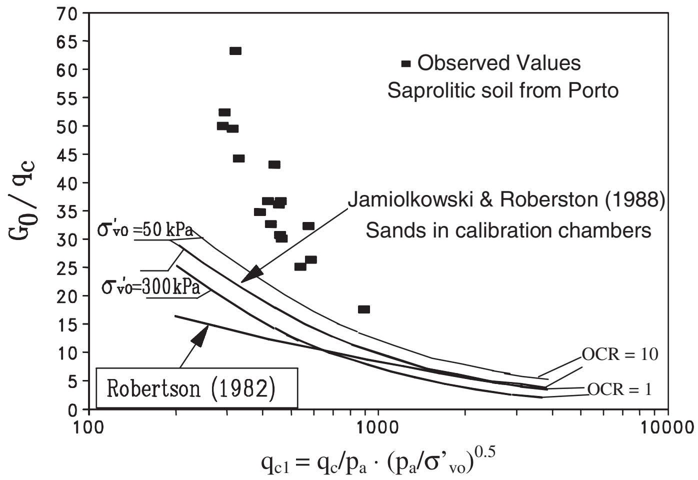


Figure 10. G_0/q_c versus $q_{c1} = q_c/p_a \cdot (p_a/\sigma'_{vo})^{0.5}$ (Robertson, 1991).

values of G_0 are substantially higher than the presented correlations, revealing a higher insensitivity of elastic properties to penetration parameters than strength.

As it will be discussed later in this paper, correlations between q_c and Young's modulus established at different stress-strain levels by triaxial tests (CID and CAD) with local strain measurement, confirmed the expected influence of non-linearity on E/q_c ratios (Viana da Fonseca, 1996), with a specific pattern.

4.7 *Pressuremeter testing (preboring, PMT, and selfboring, SBPT)*

Pressuremeter tests are increasingly considered as most convenient for soil characterization because their results can be modelled by complete constitutive laws. The self-boring pressuremeter (SBPT) allows for an insertion of the cell with minimum disturbance, making possible a rigorous theoretical interpretation of the test results and giving ideal conditions for the evaluation of geostatic total horizontal stress (Jamiolkowski & Manassero, 1996).

From a systematic analysis of PMT and SBPT (using a "Camkometer" equipment) results, Viana da Fonseca (1996) concluded that the evaluation of the coefficient of earth pressure at rest, K_0, from PMT suffers from the limitations of the pre-boring process and its determination has resulted in excessively high values (0.58). On the other hand, values of K_0 determined from very well controlled SBPT curves (the "lift-off" presssures were taken individually from each of the three strain gauges – Figure 11), ranged between 0.35 and 0.38.

High quality self-boring pressuremeter tests allow measuring horizontal stress without, theoretically, changing the magnitude of that stress, provided allowances are made for installation disturbance. So far, although it is recognised that this unchanging of in situ stresses is unrealistic, from the several testing methods, SBPT is considered a preferential method to evaluate K_0 values in geomaterials (Clarke, 1996). In the present case, the stability of the soil's structure, due its cohesive component, and the signs of good quality have enabled one to assume these values for K_0 are correct.

Some laboratory tests were conducted using high quality samples by loading specimens on triaxial chambers, with radial strain control in order to have a unidirectional vertical deformation. This was made by using hall-effects transducers. Although these processes are fundamentally non-representative of field at rest conditions, they have the great advantage of assuring a homogeneous stress field and allowing interpretation as an element with full control of the boundary conditions. The fact that most of the tested samples were taken by block trimming, from shallow depths, and carefully prepared for the test itself, and that the structural cohesive component had an important role in maintaining the soil integrity, has enabled the samples to be classified as Class1.

Viana da Fonseca (1996) has tested the same soil of the undisturbed samples – as described above – after remoulding it dynamically to the same density and moisture content. This was accomplished in a stress-path triaxial system and the specimens were instrumented radially with a Bishop type ring (LVDT). The value of K_0 obtained for the soil in the remoulded condition was practically constant and

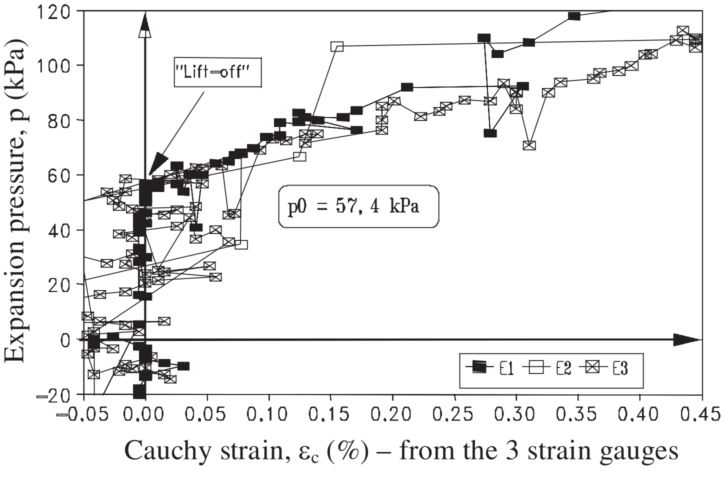


Figure 11. "Lift-off" defined on three strain gauges (120°).

equal to 0.41, while on the originally undisturbed conditions values of 0.35 to 0.38 were found within the stress range lower than the yield locus (elastic threshold). These values are coincident to those obtained in SBPT. K_0 values increase when stresses surpass the yield locus and tend to values of granular conditions (Jaky's equation). This evolution from stable to meta-stable conditions is a consequence of the progressive de-structuring up to a purely granular condition. Values of K_0 rise to 0.48, corresponding to $\phi'_{cv} = 32°$, a value confirmed in multiphase triaxial tests (Viana da Fonseca, 1996).

An FEM back-analysis of the behaviour of some sections of a carefully monitored tunnel has been presented by Viana da Fonseca & Almeida e Sousa (2001). This work has given emphasis to the importance of the assumption for K_0 and confirmed trends that point to K_0 values that are usually low for high weathering degrees (W_6–W_5 rock masses, ranging from 0.35 to 0.50), becoming higher in moderate weathering levels (W_4–W_3 masses), with K_0 values close to unity.

Jamiolkowski & Manassero (1996) remarked that the evaluation of shear stiffness from self-bored pressuremeter tests is usually adequate. The usual approach refers to the unload-reload loops, determining the corresponding modulus, as it is known that G_i – the shear modulus taken from the initial part of the expansion curve – is, even in SBPT, strongly influenced by the insertion process. The value of G_{ur} is also very dependent on factors such as creep, which is directly associated with the adopted range of the unload-reload cycle. Since the measuring system is able to overcome limitations of compliance of strain measurement and enables short but significant looping stresses to be implemented, the effects of creep are minimized (hysteresis is very low) and the value of G_{ur} represents the drained stiffness at intermediate strain levels ($10^{-4} \leqslant \gamma \leqslant 10^{-3}$), being relatively insensitive to soil disturbance caused by the probe insertion.

This matter was studied in the pressuremeter test results from PMT and SBPT. It was concluded that even on implementing unload-reload cycles, PMT do not assure enough precision to overcome limitations of strain measurement, which reveals that PMT is strongly limited for stiffness characterization purposes. The limitations were also very well defined on the pseudo-elastic *versus* virgin modulus ratios, since E_{pmdr}/E_{pm} is lower ($\cong$2) than would be expected from the ratio between E_0/E_{pm} ($\cong$18–20, with E_0 taken from G_0_CH), indicating that important disturbance resulting from the preboring process also remains. SBPT turn to be the opposite as they have proved to give a good insight into stiffness properties. Figure 12 represents one of the tests, including carefully controlled unload-reload cycles.

Taking into account the results of SBPT for evaluation of the deformation modulus, particularly of the unload-reload modulus, the ratio between G_0 and G_{ur} for the same stress levels involved in the tests were found to range between 2.6 and 3.0. These ratios are substantially lower than those reported by Tatsuoka & Shibuya (1992) for Japanese residual soils from granite ($\cong$10).

Taking the non-linearity hypothesis of Akino (1990) – cited by the previous authors – developed for a wide range of soil types, including residual soils from granite, and expressed simply by:

$$E_S = E_0, \quad \varepsilon \leqslant 10^{-4} \tag{6}$$

$$E_S = E_0 \cdot (\varepsilon/10^{-4})^{-0,55}, \quad \varepsilon \geqslant 10^{-4} \tag{7}$$

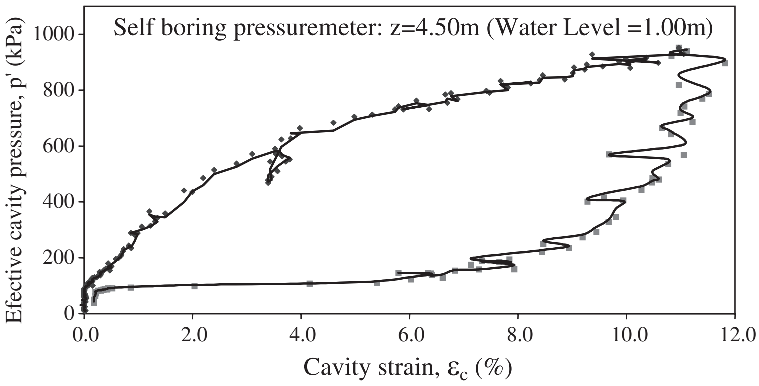


Figure 12. Example of an SBPT test with inclusion of unload-reload cycle (Viana da Fonseca, 1996).

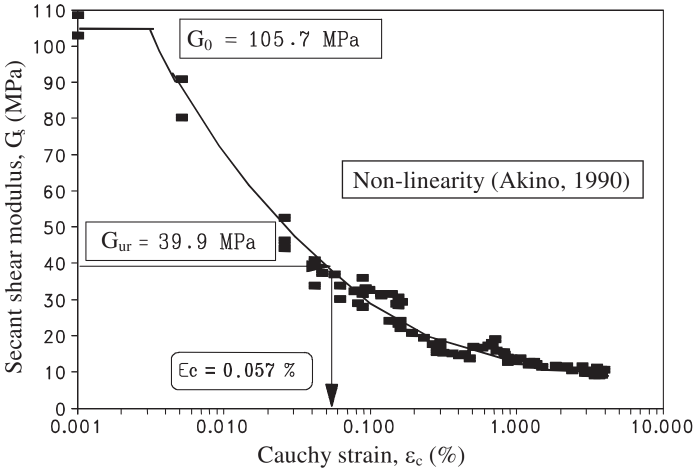


Figure 13. Non-linearity – G_{ur} over relation G_{sec} *versus* ε_c (Jardine, 1992; Akino, 1990).

the SBPT unload-reload modulus corresponds to secant values for shear strain of $5.7–7.4 \cdot 10^{-4}(6 \cdot 10^{-4})$, which agree very well with the above indicated trends. One of these values is implanted in the relation of G_s versus ε_c in Figure 13, using Jardine's (1992) model.

The evaluation of strength parameters from PMT by means of yield or limit pressures methods, leads to an underestimation of strength, even when taken as exclusively frictional. On the other hand, results from Cambridge SBPT interpreted by the Hughes et al. (1977) method (for purely frictional materials), has led to very similar values of peak angle of shearing resistance to those obtained by CPT interpretation, but there still remains the absence of an explicit cohesion evaluation. A reinterpretation of such results by recent cavity expansion theories (Schnaid & Mantaras, 1998) for cohesive-frictional materials can solve this shortcoming. Besides, a similar analysis at large deformation levels ($\psi = 0$) – as close as possible to the critical condition – of the pressuremeter curves has enabled a value of $\phi'_{cv} = 31.6°$ be defined, which is equivalent to the value obtained from specific triaxial tests (Viana da Fonseca, 1996).

4.8 *Cross-hole seismic test (CH)*

The results of cross-hole tests are very consistent and reveal a very smooth increase of G_0 with depth. In Figure 14 these variations are compared with the lines corresponding to Ishihara's (1986) proposals, for natural alluvial sands, aged and cemented, corresponding to:

$$G_0(\text{MPa}) = (7.9 \text{ to } 14.3) \cdot 0.4 \frac{(2.17 - e)^2}{1 + e} (p'_0 \cdot 10^3)^{0.4} \tag{8}$$

while for the present results the following is found:

$$G_0(\text{MPa}) = 65 \cdot \frac{(2.17 - e)^2}{1 + e} \cdot (p'_0 \cdot 10^3)^{0.07} \tag{9}$$

It can be seen that the value of the constant for the maximum shear modulus expression is much higher than the corresponding one for sandy transported soils, while the exponent m, reflecting the dependence on the octahedral at rest effective stresses, is far lower. From the reinterpretation of the observed trends – see Figure 15 – with reference to the most common correlations between N_{60} and V_s(CH) for transported aged soils (Seed et al., 1986), new values for the constants were found: fixing $\alpha = 0.84$ and $f_b = 1.3$, one gets $f_b = 1.51$.

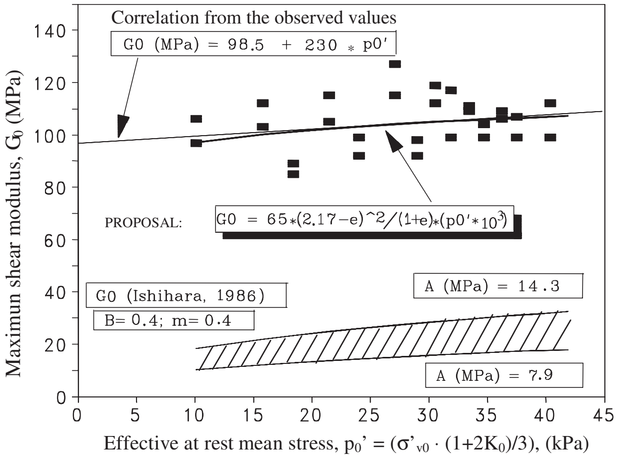


Figure 14. Comparison of observed trends of G_0 versus p'_0 and Ishihara (1986) proposals.

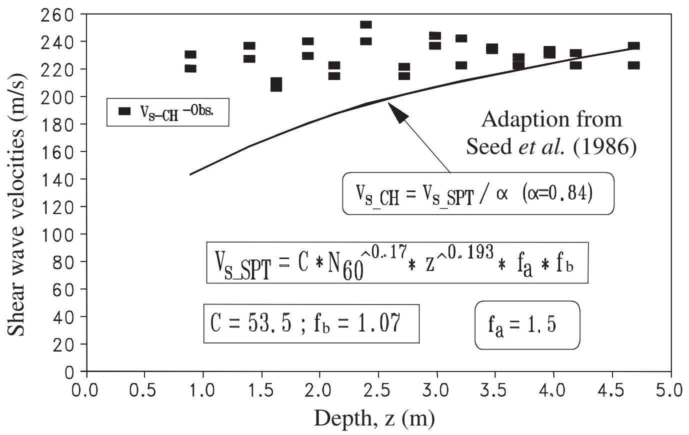


Figure 15. V_S (CH) versus V_S (N_{60}_SPT) with depth.

Reanalysing the same values with the direct correlations between G_0 and N_{60} (Stroud, 1988), the following expression was obtained:

$$G_0(\text{MPa}) = 57 \cdot N_{60}^{0,2} \tag{10}$$

confirming what has been concluded before (see 4.4) about the clear distinction dependence of both G_0 and N_{60} on σ'_{v0}, explaining such a reformulation of the correlation between them.

4.9 *Dilatometer test (Marchetti – DMT)*

Material classification in grain size terms based on Marchetti (1985) I_D index is fairly consistent (SM) with the original proposal. This contradicts the same classification when based it on CPT parameters, identifying finer groups from f_s/q_c ratio, probably due to the high percentage of laminar particles (mica) that induce higher sleeve resistances in these soils.

Campanella & Robertson's (1991) proposal for strength evaluation gives very accurate values, only if the relation between q_c/σ'_{v0} and K_D is adapted to new trends revealed in the experimental campaign, i.e.: $q_c/\sigma'_{v0} = 8.4 \cdot K_D$ against $q_c/\sigma'_{v0} = 33 \cdot K_D$, proposed by the authors for transported soils. Assuming this same ratio, the evaluation of the coefficient of earth pressure at rest, K_0, by means of K_D correlations, will be adapted to the following law:

$$K_0 = 0.736 + 0.024 \cdot K_D - 0.00172 \cdot q_c/\sigma'_{v0} \tag{11}$$

Recent trends for association of K_0 with K_D by using the state parameter, ψ (Bellotti *et al.*, 1994) were reinterpreted, resulting also in the specific relationship law:

$$K_0 = K_D/(46.8 \cdot 10^{3,82 \cdot \Psi}) \tag{12}$$

Stiffness characterization for shallow foundation settlement assessment, using the correlations between $E_{D(DMT)}$ and G_0 or $E_{s10\%}$ (Baldi *et al.*, 1989), resulted in new correlations with a much higher ratio between those parameters, than the referenced proposals:

$$G_0/E_D \cong 16.7 - 16.3 \cdot \log_{10}(p_{0N}) \tag{13}$$

$$E_{S10\%}/E_D = 2.35 - 2.21 \cdot \log_{10}(p_{0N}) \tag{14}$$

This last correlation, however, is situated between the laws that define – for those authors – the behaviour of NC and OC transported soils.

4.10 *Loading tests: large scale footing and complementary plate loading test*

A loading test of a full scale circular concrete footing, with a diameter of 1.20 m was carried out. The instrumentation used consisted of (see Figure 16): (i) 3 displacement dial gauges with 50 mm travel and 0.01 mm divisions connected to the footing to record its settlement; (ii) 9 reference pins to survey the deflection of the ground surface around the footing by means of topographic electronic equipment; (iii) 4 vertical inclinometer tubes sealed at 6 m depth to measure horizontal displacements in the ground, these tubes also being used to observe the water table.

During the test the water level remained practically constant at 1 m depth. The test was performed under drained conditions. The complete pressure-settlement curve from the footing loading test is shown in Figure 17(a). As can be seen in the enlargement of Figure 17(b), there is a clear increase of the settlement rate with load for pressure values exceeding around 125 kPa.

The time for settlement stabilisation in each load step significantly increased above this pressure. This pressure seems to represent a transition from an essentially elastic behaviour to a phase in which the cemented structure of the soil is substantially damaged, confirmed by the small difference between the inclination of the first loading curve and that of the first unload-reload cycle (Figure 17).

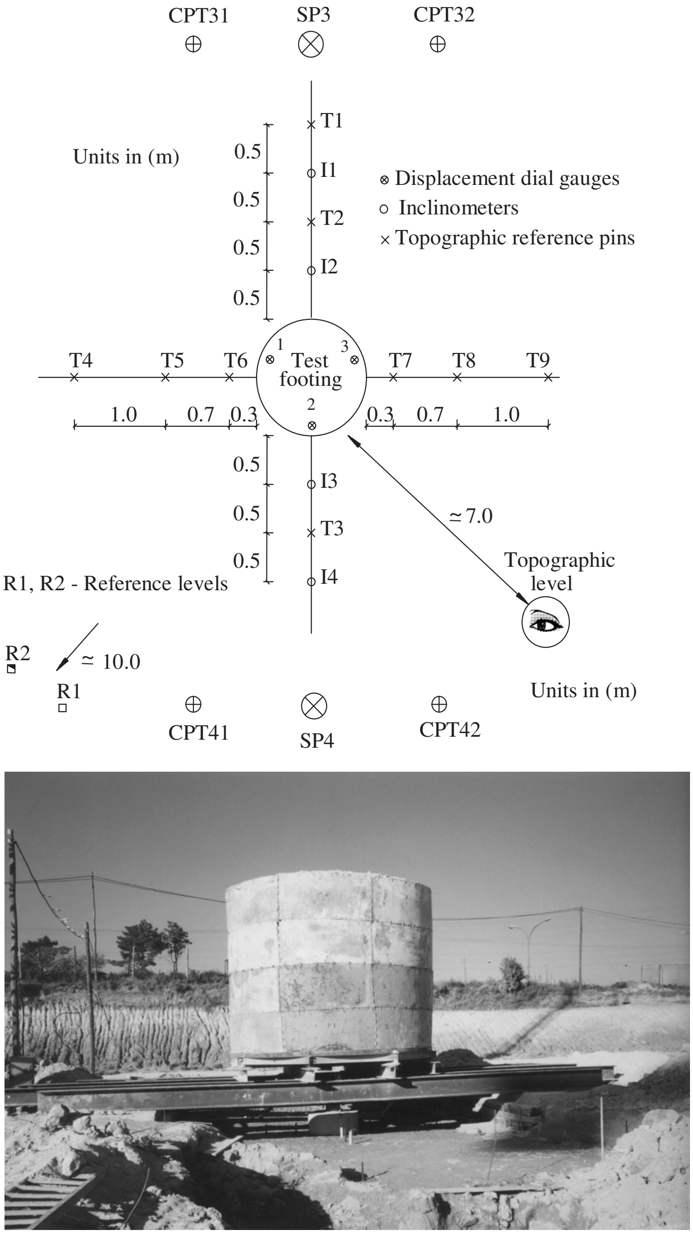
In order to establish the serviceability limit state pressure, the criterion proposed by Décourt (1992) was adopted as a reference. This stipulates that the allowable pressure on a shallow foundation on residual soil should be that which causes a settlement of 6.0 mm for a 0.8 m diameter plate loading test. This represents a settlement of 0.75% of the diameter of the loading surface, which gives a settlement of 9 mm in the present case. For the footing test curve this settlement corresponds to an applied pressure of about 195 kPa.

Viana da Fonseca *et al.* (1997) discussed the strain distribution under the centre of the footing obtained from a simplified nonlinear elastic analysis, for applied pressures of 100, 200 and 400 kPa, corresponding to a value near to the serviceability limit state pressure and to half and twice that value; strain values exceed 10^{-3} only in a restricted zone adjacent to the footing. These results corroborate evidence (Jardine et al., 1986; Burland, 1989; Tatsuoka & Kohata, 1995) of the rather low strain levels involved in a number of soil-structure interaction problems, including shallow foundations, under working conditions.

For the first load stages, the analysis of vertical displacement of six of the reference pins on the ground surface around the footing revealed an increase at an approximately constant rate after which a stabilisation occurs.

In relation to the horizontal displacements at the surface and in the ground in the vicinity of the footing, the recorded values are generally very small, most of them of the order of magnitude of the inclinometer equipment precision. For the displacements observed at the two inclinometer tubes closest to the footing (Viana da Fonseca et al., 1997), a general movement towards the loaded area was observed during the initial loading steps followed by a clear reversal of the displacement direction. Nevertheless, final displacements are still directed towards the footing.

(a)

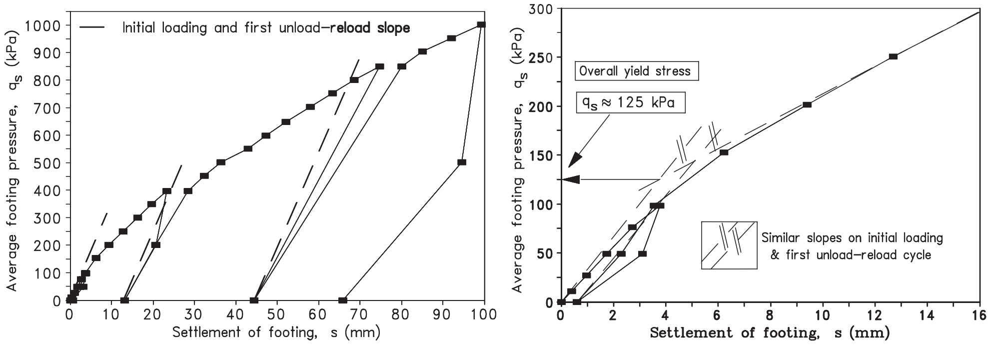


(b)

Figure 16. Plan (a) and overview (b) of the experimental site in the vicinity of loading tests.

The shape of the pressure-settlement curve is typical of a punching shear failure mode: the end of the test corresponded to a 100 mm settlement of the footing base while the ground surface pins situated at 0.3 m from its border did not reach values of settlement greater than 10 mm.

Based on the evidence presented, we can state that the applicability of elasticity theory to a settlement analysis under service conditions, is reasonable from a practical view. In residual soils the bearing capacity is due to strength factors related to friction and cohesive components of the involved ground. These should solely be determined by two or more in situ loading tests, under different sizes and over a homogeneous space.



(a) (b)

Figure 17. Pressure-settlement curve of the footing loading test: (a) general picture; (b) enlargement for $q_s \leqslant 300$ kPa.

Figure 18. Plate loading test (D = 60 cm).

The need for the execution of more than one loading test to define failure patterns has the advantage of allowing elaboration of an integrated analysis of different pressure-displacement curves, enabling the importance of the pair of factors "stiffness-depth of influence" to be studied.

On the same zone of the footing loading test, two more surfaces were prepared for testing different size plates: 30 and 60 cm diameter (the last is illustrated on Figure 18). The incremental loading steps followed the same stress levels as those of the large scale footing test. The resulting curves are plotted together in Figure 19.

A first comment is due to the fact that the normalised settlements (s/B) of the smaller plate (30 cm diameter) are substantially larger than those of the other test. This may be explained by the small scale of this plate and the inevitably induced disturbance of prepared surface levels, as well as the greater sensitivity to the presence of restricted weak zones (or macrofabric or macrostructure factors), which for foundation practice have no significance but give rise to non-normalised behaviour compared to larger scale tests.

The pressure-settlement curves present some linearity, except for the highest values of loading. It should be noted that all these tests, similarly to the footing tests, do not appear to develop a classical generalised failure surface but present a punching shear failure mode. This gives rise to typical stress increasing curves, with very smooth curvatures.

Shear strength derived from loading tests

Punching type failures in theses residual soils, give very little definition of an inflexion zone of the pressure-settlement curve. A log-log plotting can be a better way to detect it.

This has been used preliminarily to interpret the results of loading tests (Viana da Fonseca, 1996) and the following values were obtained: $B = 0.30\,\mathrm{m} \Rightarrow q_{rot} = 700\,\mathrm{kPa}$; $B = 0.60\,\mathrm{m} \Rightarrow q_{rot} = 821\,\mathrm{kPa}$;

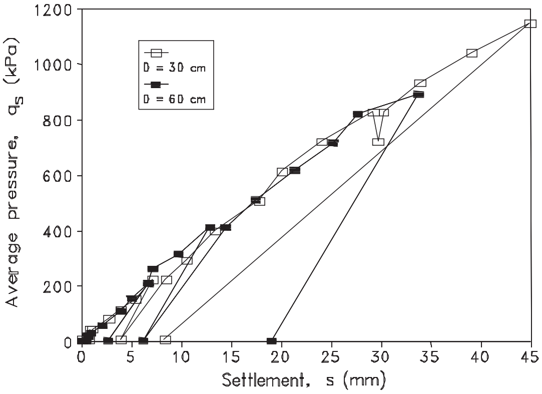


Figure 19. Pressure-settlement curve of the plate loading tests (D = 30 and 60 cm).

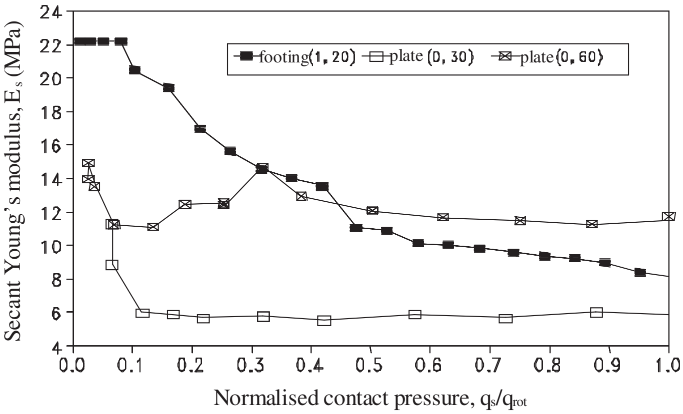


Figure 20. Young's modulus from load tests as function of the stress level (q_s/q_{rot}) – supposing stiffness constant with depth.

$B = 1.20\,\text{m} \Rightarrow q_{rot} = 950\,\text{kPa}$, q_{rot} being the "punching failure" stress, which can be distinct from the ultimate equilibrium defined by limit state analysis (Terzaghi-Meyerhof-Vésic).

If we use the values of those failure loads (details in Viana da Fonseca et al., 1997), in the bearing capacity formulation (Eurocode 7, 1994), taking account of the water level position, three equations are obtained. These can be optimised to get the two strength parameters range. The derived values were: $c' = 6.3$–$6.9\,\text{kPa}$ and $\phi' = 36.5°$–$37.0°$, revealing a fair agreement with the results obtained in extensive laboratory testing of undisturbed samples (Viana da Fonseca, 1998) and *in situ* testing (Viana da Fonseca et al., 1998).

Deformability characteristics evaluated from loading tests

The common interpretation of a set of results taken from loading tests on three distinct size load areas is made by considering Young's modulus constant with depth. A more specific analysis assuming a pattern of variation of stiffness with depth was, however, implemented elsewhere (Viana da Fonseca & Cardoso, 1999). Figure 20 presents the evolution of Young modulus with stress (strain) levels, using Boussinesq's equation for homogeneous soils:

$$E = \frac{q_s}{s} \cdot (1 - \nu^2) \cdot B \cdot I_s \qquad (15)$$

where q_s is the average contact pressure, s the corresponding settlement, B the side (diameter) of the loading surface, ν Poisson ratio and I_s a shape-rigidity factor.

Table 6 includes the Young's modulus of the soil for different load stages obtained by back-analysis of the footing loading test. This assumed a linear elastic layer with constant modulus underlain by a rigid base at 6.0 m depth loaded at the surface by a rigid footing. It should be noted that the intermediate pressure shown in the table approximately corresponds to the allowable pressure according to the criterion of Décourt (1992) for residual soils.

From the table, the trend for increasing values of the Young's modulus with the loading area is clear. Therefore, the first conclusion to be drawn is that an increase in stiffness with depth is to be expected and has an obvious relation to the results of penetration testing with depth (see Figure 6). A detailed discussion is made in Viana da Fonseca & Cardoso (1999).

Assuming elastic conditions, it is possible to evaluate the position of the so-called settlement centre, z_I (Carrier and Christian, 1973). For circular foundations, its position depends on the variation of the Young's modulus with depth and on the diameter of the footing, B. For the present case it is expected that $z_I \cong B$.

It is interesting to note that considering the results of the CID triaxial tests (details below) under a consolidation effective stress corresponding to a depth similar to the footing diameter, the tangent Young's modulus for the K_0 shear stress level was found to be $E_{tK0} = 7.8$ MPa.

The use of such a modulus in an elastic analysis of the footing loading test would lead to a crude overestimation of the observed settlement. However, if E_{tK0} is multiplied by a factor of "sampling representativeness" (G_0/G_{el}) one obtains a value that would provide a good prediction of the settlement for typical working conditions (this methodology is thoroughly exposed in Viana da Fonseca et al., 1997). This observation suggests that the application of a design methodology that corrects the values of the deformation modulus from triaxial tests by factors referenced to field tests (*Cross-Hole* tests or similar) may give good results.

Correlations between in situ tests parameters

Ratios between PMT and SPT or CPT parameters (see Table 7) do not have always correspondence, in grain size terms, with transported soils. Use is made of the similarity between N_{60}/E_{pm} values and those of Martin (1987), for Piedmont saprolitic soils.

More recently, Viana da Fonseca et al. (2001) enlarged these correlations by integrating the results of a second experimental site. Table 8 summarises these ratios.

Table 6. Secant Young's modulus, E_s, from loading test for different service levels.

	Service criterion			
Loading tests	q_s (s/B = 0.75%)	q_s/q_{rot}(*) (F_S = 10)	q_s/q_{rot} (F_S = 4)	q_s/q_{rot} (F_S = 2)
Footing	17.3	20.7	16.0	11.0
Plate (60)	11.9	11.2	12.5	12.7
Plate (30)	6.7	6.9	5.9	5.7

Notes: (*) Nearly corresponding to the allowable pressure for serviceability limit state design.

Table 7. Ratios between SPT, CPT and PMT parameters.

q_c/p_L*	f_s/p_L*	N_{60}/p_L* (MPa)	N_{60}/E_{pm} (MPa)	E_{pm}/p_L*	E_{pmur}/E_{pm}
14.3	0.390	14.6	1.4	10.6	1.4–1.9

Table 8. Ratios obtained from *in situ* tests.

q_d/q_c	N_{20}/q_c(MPa^{-1})	q_c/p_L*	f_s/p_L*	E_{PMT}/p_L*	P_{0DMT}/P_{0PMT}	P_{1DMT}/P_{fPMT}	E_D/E_{PMT}
0.75–1.25	0.6–0.8	4–6	0.10–0.25	12	2–3	≅1	≅1.5

5 ON THE SPECIFIC APPLICATION OF FOOTING SETTLEMENT PREDICTION METHODS BASED ON *IN SITU* TEST

Some solutions from the literature that use SPT parameters for settlement evaluation, were tested and the following was concluded:

(i) Terzaghi & Peck proposal led to settlements 2 to 4 times higher than observed;
(ii) Parry's (1978) proposal, taking $\alpha = 0.3$, has given reasonable results for the very early load levels (up to 20% of failure – before yield locus – definition in Viana da Fonseca et al., 1997), but is strongly unconservative for higher load levels;
(iii) Burland & Burbidge (1985) proposal ($\alpha \cong 1.71$) is roughly conservative, with values of predicted settlements 2 to 3 times higher than the observed ones (for loads up to serviceability limits, adapted from the Boston code: $s/B = 0.75\%$); a lower value for $\alpha = 0.855$, is in accordance with similar trends in Brazilian residual soils (Rocha Filho, 1986).

From the CPT based semi-empirical solutions for settlement evaluation, Schmertmann et al.s (1978) method was tested with great accuracy (fine layer discretization) for the most representative PLT ($D = 0.60$ and 1.20 m). An excellent reproduction of the observed curves was obtained (even in non-linearity terms) when the values of $E/q_c = \alpha$ are modified to 4.0 to 4.5, higher than those proposed by the authors for sandy soils.

The most adapted Ménard's rheological factors ($\alpha = E/E_{pm}$) for correction of PMT modulus in order to get the best convergence between observed settlements in PLT tests (in serviceability load levels) and calculated by means of the classical elastic solution taking into account the concept of settlement centre (Viana da Fonseca, 1996), was found to be typical of silty soils ($\alpha = 1/2$), corresponding to the actual grain size distribution of this saprolitic soil. The use of PMT unload-reload modulus happens to give the direct values of the Young's modulus to be taken in the same solutions. On the other hand, the values of SBPT unload-reload modulus reproduce the behaviour of intermediate cycles in PLT tests.

Finally, a load-settlement analysis of the most significative PLT, similar to CPT interpretation but using DMT Modulus (E_D) was made. The non-linear methods from Leonards & Frost (1988) – based on Schmertmann's influence diagrams – and Robertson (1991) were used and the best adjustment of the experimental results was obtained for a factor of $E/E_D = 2.34$, which is an intermediate value between NC and OC sandy deposits (Berardi et al., 1991). The non-linearity of both PLT curves ($D = 0.60$ and 1.20 m) was also reproduced.

A more detailed analysis of some approaches based on the Theory of Elasticity was developed in another paper (Viana da Fonseca, 2001), with especial emphasis on semi-empirical methodologies based on results of SPT, CPT, PLT and triaxial tests on high quality samples with the results from local instrumentation. Some of the well established methods (Parry, Burland and Burbidge, Anagnastopoulos et al., Schmertmann et al., Robertson, Ghionna et al. and Wahls and Gupta) were tested and some parametrical and methodological adaptations were suggested that best fit the observed behaviour.

An analysis of the results of the footing test was conducted by reference to the serviceability limit state criteria referred to above (Décourt, 1992). From this load value, which corresponds to a settlement of 0.75% of the loading area diameter, and the loads corresponding to different global safety factors towards bearing capacity failure, it is possible to calculate through elastic solution the related secant modulus (included in Table 9).

Considering the increase of q_c from CPT with depth (see Figure 6b), a convergence analysis, based on an elastic solution, was made by accepting the settlement centre concept. The procedure was based on the proposal of Burland & Burbidge (1985) for evaluation of the the depth of influence as a function of the degree of non-homogeneity, E_0/kD.

Accepting the proportionality between the design modulus E and q_c, expressed by α ($=E/q_c$), the variation law of the modulus with depth will be expressed by:

$$E\ (\text{MPA}) = E_0 + k \cdot z = \alpha \cdot q_c = \alpha \cdot (4.68 + 1.47 \cdot z,\ m) \quad (16)$$

Table 9. Secant deformability modulus back-calculated from the footing load test.

Load criteria	$s/B = 50.75\%$	$F_s = 10$	$F_s = 5$	$F_s = 2$
E_s (MPa)	17.3	20.7	17.5	11.0

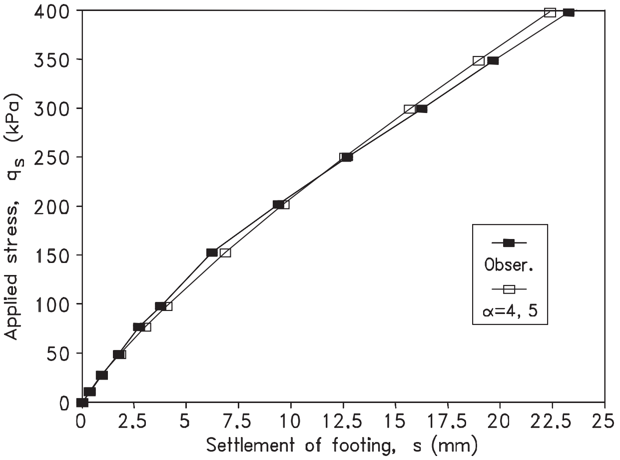


Figure 21. Settlement prediction of the footing load test using CPT results and adapting Schmertmann et al. (1978) coefficients.

Therefore, it is feasible to deduce the degree of inhomogeneity, E_0/kD, which enables oneto determine the position of the settlement centre from Burland & Burbidge's chart. Associating the value of q_c for that depth with the secant deformability modulus back-analysed from the footing load test, the value of α becomes equal to 3 and 4 for global safety factors of around 5 and 10, respectively. The former value ($\alpha = 3$) is very consistent with the serviceability limit criteria, although it can involve significant plastification in the ground (confirmed by numerical elasto-plastic analysis).

The method of Schmertmann et al. (1978) for settlement evaluation was considered, combining the proposed strain influence factor diagrams with the variation of E over depth. It has to be noted that, this approach introduces a non-linearity on stiffness, in spite of being based on an unique equation for E. This formulation was applied to the footing load test results, considering moderate stress levels (F_s around 2.4, $q_s = 400$ kPa) and revealed excellent convergence between calculated and experimental load-settlement curves for α equal to 4.5, with the actual non-linear response amazingly well recovered (Figure 21).

This value of α, which is somewhat higher than the ones commonly considered for normally consolidated sandy transported soils, is certainly due to natural structural factors, associated with the relic interparticle cementation and fabric of residual soils. Robertson & Campanella (1988) had already stated that α values could be as high as 3.5 to 6.0 or 6.0 to 10.0, for aged normally consolidated and highly overconsolidated sandy soils, respectively. On the other hand, Brazilian conclusions on the coefficients to be applied to elastic or pseudo-elastic solutions for settlement evaluation in residual soils are in strict accordance with these pieces of evidences. Rocha Filho (1986) applied the authors proposals to the results of loading tests on shallow foundations and plates with diameters from 0.40 to 1.60 m, carried out on residual soils from gneiss in the university campus of PUC in Rio de Janeiro, resulting in ratios of calculated to observed settlements of between 1.5 and 2.5. The ratios obtained in this study are even larger (2.7–3.4).

6 LABORATORY TESTING FOR MECHANICAL CHARACTERISATION

6.1 *Note on sampling*

Laboratory tests for mechanical characterisation were carried out on undisturbed samples taken from large blocks trimmed in open ditches at a depth of 0.5 to 1 m below the level of the referred footing base (this is situated 1 m below the natural surface). All the sampling and handling procedures were undertaken with the utmost care in order to preserve the natural structure of the saprolitic soil. It has to be emphasised that this sampling was undertaken near the surface and that implies little effects of stress relief, preserving in principle the integrity of the natural structure. Taking into account, on the one hand, the homogeneity of the ground underneath the surface to depths of about 6 m and, on the other hand, the conclusions on induced strains that have just been presented, the above mentioned blocks can be taken as representative of the saprolitic soil to be analysed.

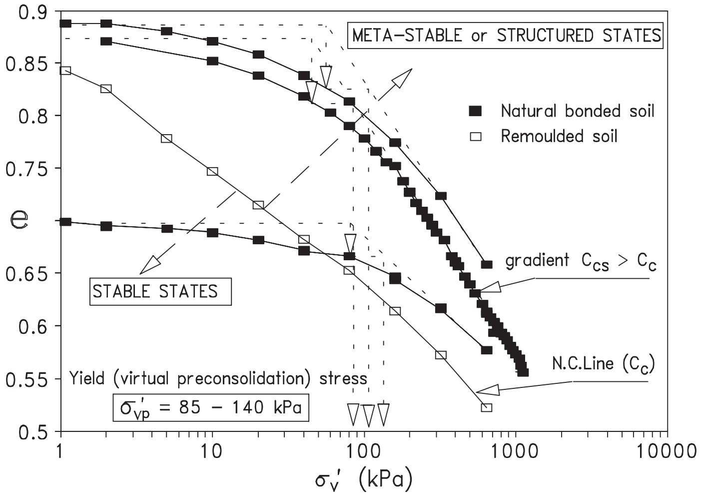


Figure 22. Oedometer curves – state definition.

The influence of the dimension of tested specimens was studied. It was concluded that it should not be strictly and solely determined by common rules of the ratio between grain size and diameter (height) of the specimen but mainly by structure and fabric conditions.

Oedometer and triaxial specimens were obtained by driving rings or split tube samplers into the blocks with the help of a static thrust. The requirements of GCO (1990) concerning the cutting shoe, the wall friction (smooth inox steel faces) and the use of a non-return valve have been adopted. It is recognised that tube sampling may significantly reduce stiffness and strength of soils (Hight, 1993, 1996, 2000 and Hight et al., 1992) and a systematic research on the influence of sampling methodologies on residual soils is being carried out in the University of Porto.

6.2 *Oedometer and isotropic consolidation tests*

Figure 22 includes the results of oedometer tests revealing three distinct zones typical of cemented soils (Vaughan, 1988) and corresponding to different states and ranges of the compressibility index: (i) "stable", C_r = 0.007–0.018; (ii) "metastable", C_{cs} = 0.129–0.289; and (iii) "granular or de-structured", C_c = 0.101–0.172.

In the first state the soil generally preserves its natural cemented structure, while in the third state this structure is completely destroyed; the intermediate state is associated with progressive de-structuring, resulting from the gradual breakage of inter-particle bridges by compression.

While the first zone can be likened to an overconsolidated state in transported soils, the second, although apparently similar to a normally consolidated state, is really a transition state characterised by higher values of compressibility than the same material would have under remoulded conditions (granular state). Results for the remoulded soil are included in the figure.

A virtual preconsolidation stress, σ'_{vP}, can be defined as the value that separates the first two zones, where most of the stress states generated by current loading conditions are situated. For that purpose the method proposed by Pacheco e Silva (1970), modified by Décourt (1992) was used, since it has demonstrated a good reproducibility in residual soils. The method is illustrated in Figure 23.

It can be described follows: (a) C_c is determined by a linear regression between e and log p (it is recommended that for this first regression the values of *p* considered should be at about 50% higher than the expected σ'_p and lower than ten times σ'_p); (b) entering with the initial void ratio e_0 in the first regression, a pressure is determined; (c) a second regression between *e* and *log p* is now established using the pressures immediate by above and below the pressure determined in "b" (although being aware that it is a curve rather than a straight line defines the variation of the void ratio as a function of the log of the pressure in this range, the differences are considered negligible); (d) entering with the pressure determined in "b" in this second regression, a void ratio is determined; finally, (e) entering with this void ratio in the first regression, the pre-consolidation pressure (σ'_p) is determined.

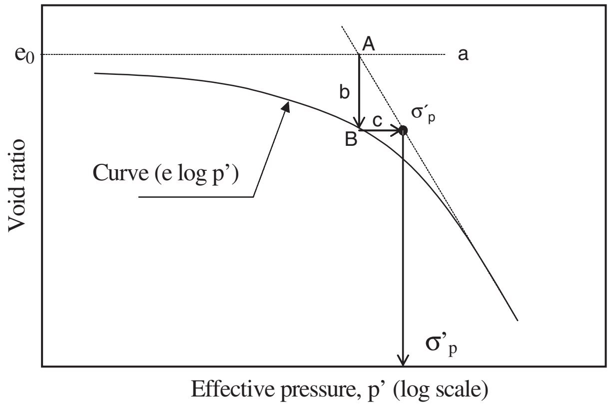


Figure 23. Modified Pacheco e Silva method (Décourt, 1992).

The values obtained for apparent preconsolidation stress were $\sigma'_{vP} = 85–140\,kPa$, more than 4 times the natural at rest vertical effective stress.

Values of C_r and C_c from the tests are reasonably consistent with those commonly reported in the literature for similar soils (Lacerda et al., 1985). Nevertheless, the values of the tangent drained constrained modulus calculated for the same consolidation effective stresses are substantially lower than the corresponding tangent Young's modulus obtained from triaxial tests (described in the following section), after taking into account the effect of Poisson's ratio. This may be due to the height of these specimens being significantly smaller than that of the triaxial specimens, leading to greater damage during preparation and handling. At small strains, bedding and contact problems may also have contributed to that trend. Due to the fact that in residual soils there is a gradual evolution (degradation) of stiffness due to progressive destructuring, it may be advantageous to define an unique law for the evolution of the unidimensional compression modulus (D) with applied stress. This leads to the following expression:

$$D(=1/m_v) = 2536 \times 10^{(9.1\times10^{-4}\times\sigma'_v)}(\mathrm{kPa}) \tag{17}$$

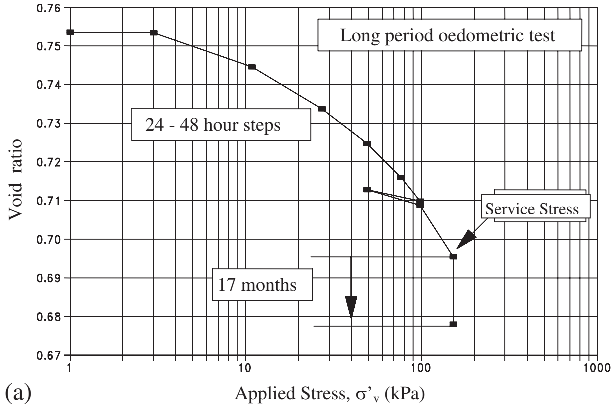
Problems related to primary consolidation are generally negligible in these residual soils (values of c_v between $0.2–4.0 \times 10^{-4}\,cm^2/sec$), however problems can arise in long term loading situations, as for regional experience based on the analysis of old building settlements.

For this reason, the importance of secondary compression, and generally creep, of this soil was analysed by testing in oedometric conditions, a few undisturbed samples over a long period of time at different loading levels. Values of the coefficient of secondary consolidation for loading periods no longer than 7 days (usually these tests do not extend for much longer periods) and for stresses lower than "aparent preconsolidation stress" are modest – lower than 0.003 (typical of overconsolidated soils). The yield potential increases rapidly beyond that stress, because of progressive de-structuring, but c_v values do not get much higher than those obtained for similar stresses over remoulded samples.

Amar et al. (1994) reported evidence for an increase in the rate of settlement with time (creep) of experimental footings on sandy natural deposits. These results led us to study this problem by keeping a few undisturbed samples of 125 mm diameter loaded for long periods. Figure 24 illustrates the results of one of these tests where the applied stress level was in excess of σ'_{vP}.

There appears to exist two distinct zones with a quasi-linear response in semi-log time scale with rather different values of the creep ratio, translated as c_α. Between these two zones there is a convenient smooth transition in time that ranges between 80 to 90 days. This transition has to be related to a progressive time dependent yield phenomenon. Secondary consolidation values become significant – $c_\alpha = 0.0135$ – and very important for foundation design purposes.

These high c_α values could be explained by the fact that after the loss of inter-particle bonding the presence of a significant percentage of micaceous particles with high slickenside potential determines the strong development of these deformations. Collapsibility of these soils was estimated by multistage oedometric testing of natural moisture state specimens, incrementally saturated with stress levels below and above the apparent preconsolidation stress (with a hydraulic cell with 125 mm diameter),



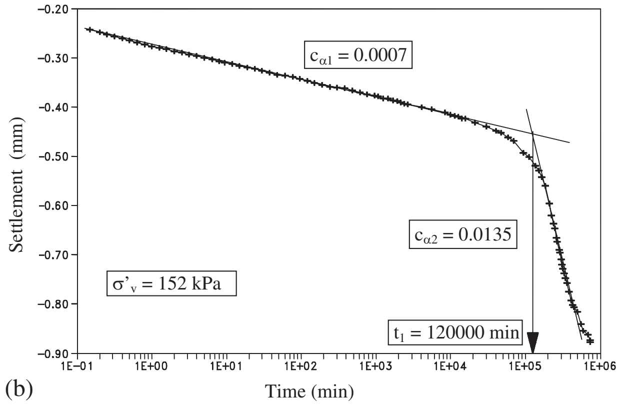


Figure 24. Long period oedometric test (a) compressibility curve; (b) consolidation curve.

concluding that this soil is moderately expansive for low stress levels and moderately collapsible for high ones.

Isotropic consolidation tests with local measurement of axial and radial strain provided values of the apparent isotropic preconsolidation stress of $\sigma'_{mP} = 40$–$60\,kPa$. This range is slightly lower than the one deduced from the oedometer tests: considering $K_0 = 0.38$ and taking the values of σ'_{vP} indicated above ($\sigma'_{mP} = 50$–$80\,kPa$). This conclusion corroborates the hypothesis of Vaughan (1988) that yield surfaces for volumetric compression are anisotropic and may be centred on the K_0 stress axis (Viana da Fonseca et al., 1997).

6.3 *Triaxial tests*

A large number of triaxial tests (43) was performed with different specimen sizes, consolidation stress conditions and stress-paths.

Recognising the importance of local strain measurement techniques in triaxial testing, special care was taken over this subject. This is particularly important for highly structured sensitive soils, with high values of initial stiffness and subsequent pronounced non-linear stress-strain responses. The local strain was measured with a pair of axial strain inclinometers from the Imperial College of London. Figure 25 compares stress–strain curves from one of the tests obtained by classical (external LVDT) and local strain measurement systems. The former technique leads to unrealistic stiffness values.

Some specimens were tested at natural moisture condition (associated with high values of degree of saturation, the most common between 80–100%, these later corresponding to the best conditioning processes) and others, taken from the same samples, previously saturated by applying 10 kPa increments of back and confining pressures – maintaining an effective isotropic stress of 10 kPa – up to values of

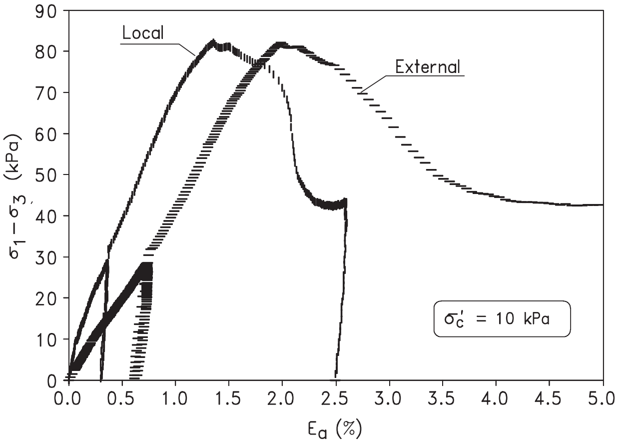


Figure 25. CID triaxial test: stress–strain relations with local and external strain measurement.

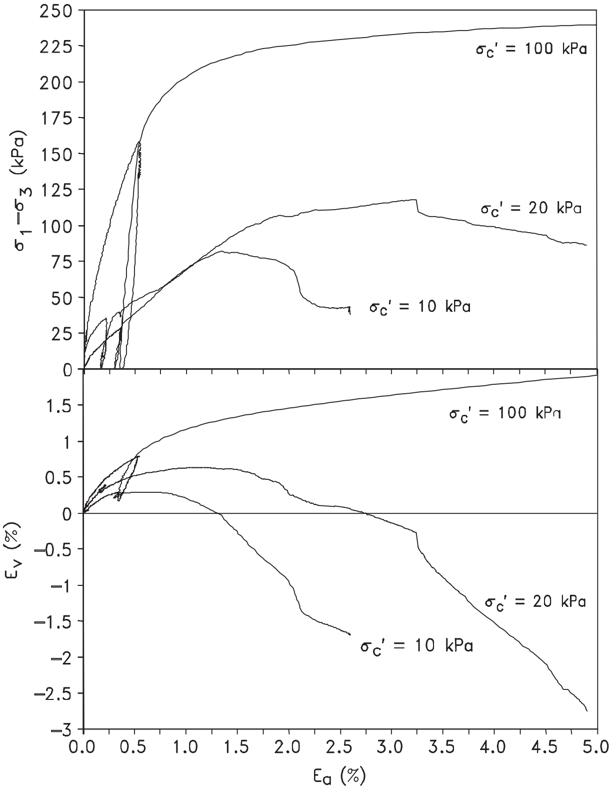


Figure 26. Results from CID triaxial tests under three distinct consolidation stresses.

300 kPa to achieve full saturation. Differences in behaviour (in CID and CK_0D tests) between these two preparation patterns were not clear and results from the two methods have not been distinguished.

To fulfill the requirements of drained shear conditions the rate of deformation was 0.002 mm/min. Intermediate unload-reload cycles were performed starting at low to moderate stress levels. An example of one of this set of results of CID triaxial tests is presented in Figure 26, where the curve for the local instrumentation results is included together with the curves from other tests with similar measurements, but different consolidation stresses.

Deformability

Figure 26 shows that the soil exhibits substantial brittleness for the lowest effective consolidation stress, while for the others the brittle behaviour tends to vanish. Thus, for consolidation stresses much higher than the at rest stress state, the stress-strain response tends to be typical of de-structured materials. For the lower consolidation stresses (10 and 20 kPa) peak values of deviatoric stress are mobilised before the highest value of the dilatancy ratio is reached.

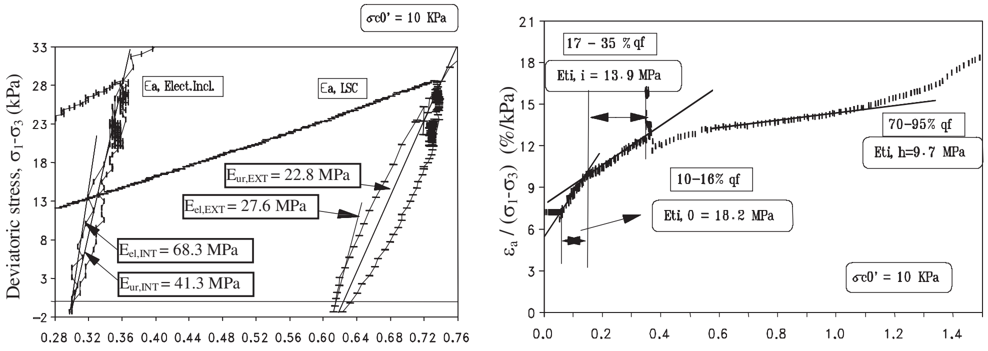
This indicates that the mechanical behaviour of the saprolitic soil is controlled by cementation between particles rather than by dilatancy phenomena related to particle interlocking. This latter type of behaviour, typical of dense granular transported soils, is not compatible with the saprolitic soil's fabric which exhibits a low to medium density (e = 0.60–0.85) for an essentially sandy soil.

In order to study the dependency of stiffness on the effective consolidation stress, several types of Young's modulus were considered: (i) "pseudo-elastic", E_{el}, deduced from the initially linear reload branch of an intermediate unload-reload cycle, (ii) "unload-reload", E_{ur} deduced between vertices of the cycle, (iii) initial tangent, E_{ti}. This latter modulus, as it was deduced from a classical hyperbolic matching of the stress-strain curve, was considered in three levels. Designated by $E_{ti,0}$, $E_{ti,i}$ and $E_{ti,h}$, these moduli have taken into account, respectively, the lowest shear stress values – where the natural bonding between particles is mostly preserved (inside the elastic yield locus), the intermediate states – representative of the metastable condition, where progressive de-structuring is observed – and finally the last is taken from values of shear stress between 70 and 95% of failure and where the behaviour is significantly of granular type. Figure 27a illustrates these different trends and adopted options.

The importance of the use of local (internal) strain instrumentation is global for the evaluation of deformability characteristics. As an example, unload-reload cycles for low stress levels (most inside the "yield locus" -where very low plastic energetic dissipation phenomena are expected) can be compared when registered from local strain measuring devices. The difference in results, as illustrated in the figure, and the quasi-elastic pattern of behaviour, solely evident in local instrumentation (presenting very incipient hysteresis), proves the importance of these devices.

This is also clear in the difference between the constitutive answers, being most notorious for the low levels of confinement stress and emphasizing the importance of very small deformations in materials where the normalised stiffness (E/σ'_{oct}) is high, a characteristic of natural structured soils or weak rocks. This difference is also accentuated in the values of modulus that are mostly influenced by the bonded structure: the elastic (initial or dynamic) or other intermediate moduli, such as "pseudo-elastic", unload-reload and initial tangent. Values of the modulus for very low stress levels, measured using local strain devices, are largely dependent of the confinement stresses revealing a large resistance to shear stresses when the soil is loaded under confinement stresses inside the yield locus.

This trend corroborates similar stability trends detected in the variation of G_0 (CH) with depth (Figure 6), a sign of a certain constancy with the effective confinement stress.



(a) Axial strain, εa (%) – local (INT) and external (EXT) measurement

(b) Axial strain, εa (%) – Local measurement

Figure 27. Young modulus from triaxial test with local (internal) and external strain measurement: (a) pseudo-elastic, E_{el}, and unload-reload, E_{ur}; (b) hyperbolic: $E_{ti,0}$, $E_{ti,i}$ and $E_{ti,h}$.

Table 10. Janbu (1963) stress dependency parameters for deformability modulus.

	E_{el} (pseudo-elastic)		E_{ur} (hysteresis)		E_{ti} (70–95% q/q_f)	
Parameter	*INT. INS.*	*EXT. INS.*	*INT. INS.*	*EXT. INS.*	*INT. INS.*	*EXT. INS.*
K	2074	1139	1228	813	364	272
n	0.533	0.642	0.551	0.693	0.574	0.570
(*r*)	0.917	0.943	0.914	0.930	0.661	0.606

E_{el}; E_{ur}; E_{ti} different types of Young's modulus considered and the deduction of which are explained in the text;
INT.INS.; *EXT.INS* values deduced from triaxial tests with local/internal or classical/external strain measurement, respectively.

To make these trends clear, Table 10 presents the values of the stress dependence parameters obtained in the laboratory from Janbu's (1963) law:

$$E = K \cdot p_a \left(\frac{\sigma'_c}{p_a}\right)^n \tag{18}$$

Values of *K* and *n* deduced for the above-defined modulus and calculated from the results taken from local (internal) or external strain measurement are compared. These sets give a very clear idea of the importance of the instrumentation in triaxial testing. Values of *n* obtained from local/internal strain measurement ("*INT.INS*") are systematically lower than those taken from external devices ("*EXT.INS*"), and most significantly are parallel with what is observed in the variation of G_0(CH) with depth.

On the other hand, values of *K* are significantly higher when evaluated by "*INT.INS*" than classically. By introducing the notion of "*instrumentation ratio*" we can estimate the factors for correcting common laboratories results (usually using classical external devices). For the present study, it was concluded that the ratio n_{int}/n_{ext} seems to be more dependent on the type of modulus, although for practical reasons and because of the closeness of the obtained experimental results, unity can be assumed. On the other hand, the ratio K_{int}/K_{ext} values is more stable and its value close to 2, confirming the importance of these new instrumentation techniques in reproducing the absolute values of natural stiffness.

It is interesting to note that a non-linearity analysis by Jardine et al. (1986), has revealed a fairly good reproducibility of the triaxial tests results (Figure 28). On this saprolitic soil, the parameters deduced for this model (see Table 11) indicate a much more pronounced non-linearity than corresponding transported soils, even when highly overconsolidated. This is a consequence of the presence of bonding bridges in this residual soil.

Strength

Strength parameters obtained from laboratory tests showed a good consistency. Ranges of 37° to 38° for the angle of shearing resistance and 9 to 12 kPa for the effective cohesion were obtained.

Table 12 summarizes the results obtained from the extensive experimental campaign on undisturbed and remoulded samples, using several types of laboratory equipment as well as some diversity of specimen conditions and methods (stress paths).

Concerning to the peak strength values, it has to be emphasised that the best regressions were obtained from the specimens of 100 mm diameter. Values taken from tests with small specimens (diameters no greater than 50 mm) were rather disperse (Figure 29). The critical state line was defined with great care by testing nine remoulded specimens a three distinct densities and consolidation stresses (Viana da Fonseca, 1996).

The value of the critical state angle of shearing resistance was deduced: $\phi'_{cv} = 31.6°$. Multi-cycling direct shear tests mainly allowed the lowest value of the angle of shearing resistance for large deformations ($\phi'_{cv} = 30.9°$) to be defined. This is associated with the residual strength concept.

Comparison with in situ testing results

Values of the angle of shear resistance deduced from the best known correlations with in situ testing for sandy transported soils (De Mello, 1971; Robertson & Campanella, 1983; Décourt, 1989) were the following (details in Viana da Fonseca et al., 1997):

– SPT: from the values of $(N_1)_{60}$, $\phi'_{avg} = 38°$;
– CPT: from the ratio q_c *versus* σ'_{v0}, $\phi' = 44$–45°.

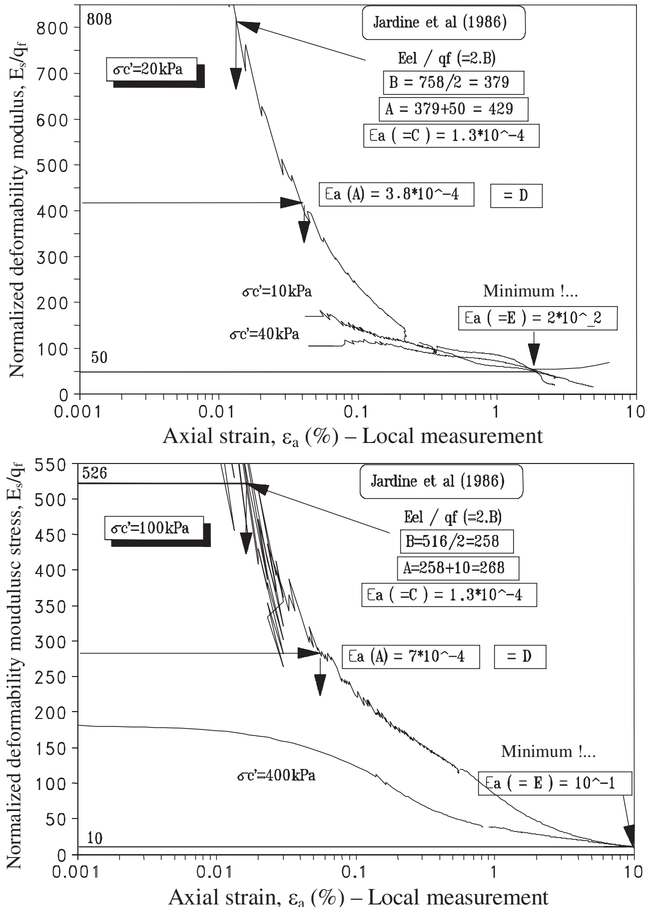


Figure 28. Non-linearity by Jardine et al. (1986) model.

Table 11. Parameters obtained for the saprolitic soil – Jardine et al. (1986) model.

Soil	Conditions	A	B	C(%)	α	γ	$\varepsilon_{máx} \equiv E(\%)$
Saprolitic soil from granite	Undisturbed: $\sigma'_c = 20\,kPa$	429	379	0.013	2.212	0.4482	2
(drained conditions)	Undisturbed: $\sigma'_c = 100\,kPa$	268	258	0.013	1.840	0.5048	10
Jardine et al. (1986)	Remoulded (R1)	850	1000	0.008	2.023	0.5943	0.2
Clays (non drained)	Undisturbed & remoulded (R2)	1420	1380	0.009	2.098	0.5050	1.5

Parameters A, B, C, α, β and E state for the non-linearity of the periodic logarithmic function.

There is a large discrepancy between these proposals. This reflects the simultaneous sensitivity of q_c to frictional and cohesive components. As referred above, the angle of shearing resistance has to be taken as a secant value on the stress space and, consequently, this latter parameter decreases with the increase of the vertical effective stress. This is very well defined on the results plotted in the formerly referred abacus (Figure 30).

Nevertheless, care should be taken with the adoption of this solely frictional resistance for the evaluation of the ultimate load of shallow foundations, since it is well recognised that cohesion has a strong influence on the values of calculated q_{ult}, as well as the strong non-linear increase of the values of bearing capacity factors with increasing values of ϕ'.

Table 12. Strength parameters obtained from laboratory testing (Viana da Fonseca, 1996).

Failure mode	Test description					φ' (°)	c' (kPa)	φ' (°) forcing c' = 0
Peak values $(\sigma'_1/\sigma'_3)_{max}$ or $(\sigma_1-\sigma_3)_{max}$	CID/CK_0D Triaxial	Compression	p'_f< 100kPa	Classic compression with specimens of 100mm diameter and internal strain measurement		36.8	11.7 (r = 1.0)	43.3 (r = 0.90)
				All compression test, including conventional p'=const.) with & without local instrumentation		37.2	8.7 (r = 0.98)	43.3 (r = 0.90)
			p'_f< 70kPa	All compression test for low values of confinement stresses (structure is preserved for the adopted depths of sampling		38.0	8.6 (r = 0.84)	46.4 (r = 0.80)
		Extension	p'_f< 70kPa	Lateral compression, axial extension and p' = constant with decreasing q		21.4	22.0 (r = 0.86)	—
	Simple shear test		σ'n 30, 75 kPa	Only two values were significative (drained tests without measuring the lateral stresses – in the involving rings		38.5	2.8 (σ = 38°)	—
	Direct shear test	Jewell symetric box (100x100x30)		σ'_n < 40 kPa		42.6	2	—
				σ'_n < 150 kPa		41.6	5	—
		Jewell symetric box (60x60x20)		σ'_n < 150 kPa		35.8	21	—
Ultimate values	Critical	Triaxial tests with Remoulded samples		Special procedure for definition of the steady stable line 9 specimens wiht e_0 between 0.54 and 0.85 and stresses of: σ'_c (CID/CIU) = 10,40,100,300 kPa		31.6	—	31.6 (r = 0.98)
	Residual	Direct shear test		Jewell symetric box (100x100x30) σ'_n= 10,20,40 kPa Undisturbed and remoulded samples after 2 reverse cycles	From the ψ value (Taylor)	30.9	—	30.9

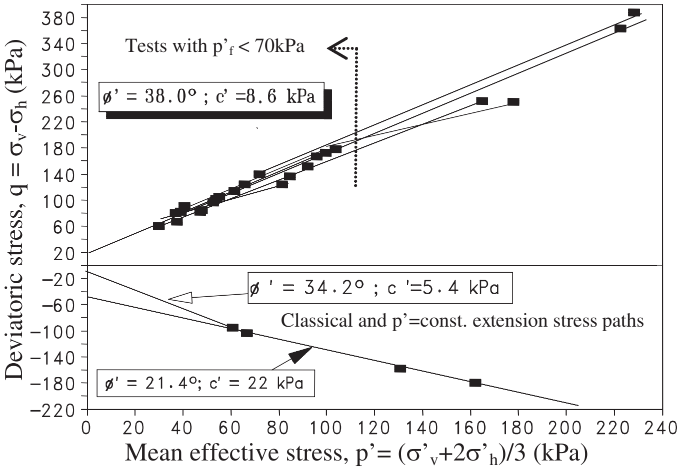


Figure 29. Failure envelopes from triaxial tests in compression and extension on undisturbed samples.

Shear strength parameters deduced from the results of Ménard pressuremeter tests results – even using fairly distinct interpreting methodologies, such as yield stress and limit pressure criteria or even the dilatant theory or the semi-empical proposals of Baguelin et al. (1978) – are rather inconsistent amongst themselves and incongruent with other tests deductions (see above). This gives rise to a strong appeal for making new efforts towards reinterpreting these tools specifically for residual soils. This is important for the fact that we are dealing with cohesive-frictional materials, requiring special modelling methodologies of pressuremeter tests (Schnaid & Mantaras, 1998).

On the other hand, results from the Cambridge SBPT interpreted by the Hughes-Wroth-Windle method (for purely frictional materials), has led to very similar values of peak angle of shearing resistance to the ones obtained by CPT interpretation ($\phi' = 40.0$–$47.7°$), and these remains the lack of an explicit cohesion

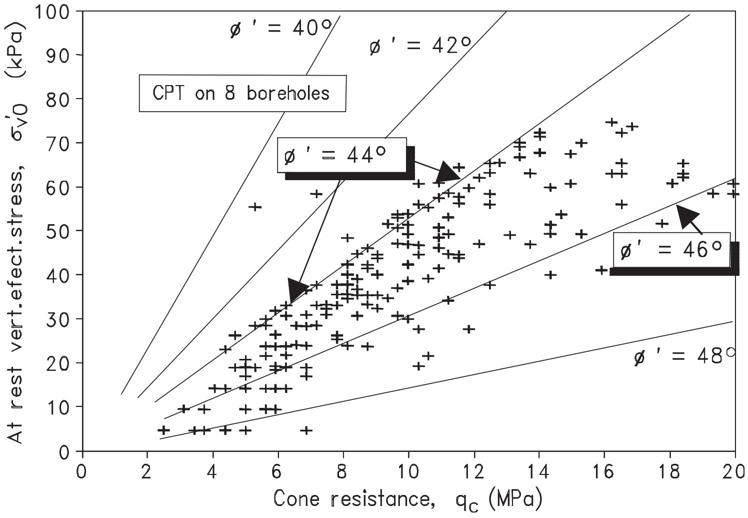


Figure 30. Evaluation of ϕ' by means of q_c versus σ'_{v0} (Robertson & Campanella, 1983).

determination. Nevertheless, if we make the approach of the failure points taking into account the constant value of at rest effective vertical stress, we obtain the following parameters: $\phi' = 26.4°$; $c' = 20.8$ kPa ($r = 0.976$). These are closer to the parameters that define the failure envelope in extension stress-path triaxial tests on undisturbed samples (Table 12 and Figure 29). This proves the urgent demand for a more consistent and systematic research into the importance of the stress path on the definition of failure envelopes and the comparison of the failure parameters from different testing methods.

The use of DMT for estimating the angles of shearing resistance, ϕ', evaluated from the diagram of Campanella & Robertson (1991) – developed from Marchetti (1985) – resulted in values between 40° and 42°, which are situated between those taken from triaxial tests and from the SBPT. The fact that the K_D parameter reflects both the influence of the friction and cohesive components can explain the higher values of ϕ' (DMT) values in relation to the main sets, as those deduced from triaxial tests. Cruz et al. (1997) suggest a method for identifying effective cohesion from DMT together with a single set of compression triaxial tests on undisturbed samples.

Finally, if we include the values of failure loads on PLT tests on circular rigid plates of 30 and 60 cm diameter and on a concrete footing of 120 cm diameter (detailed description in Viana da Fonseca et al., 1997), respectively, $q_f = 700$, 821 and 950 kPa, and if we use the bearing capacity equation taking account of the water level position, we obtain three equations. These can be optimised to get the two strength parameter ranges. The values obtained were: $c' = 6.3$–6.9 kPa and $\phi' = 36.5°$–$37.0°$, revealing a fair agreement with the previously dscribed laboratory results and *in situ* testing interpretations. It must be emphasized that failure on these residual soils is usually of punching type, with very little definition of an inflexion zone of the load-settlement curve. A log–log plotting can be a better way to detect it.

7 CONCLUSIONS

This paper has presented some evidence of the particularities, in geotechnical terms, of young residual soils (saprolitic) from granite that dominate most geotechnical profiles in Porto, Northern Portugal. Specific concepts for classification, where chemical and mineralogical indices can represent determinant tools, here complemented by physical indices, have proved to be very useful. Micro and macrofabric analyses, integrated with decomposition and lixiviation indices, have allowed the expectancy of a more or less pronounced metastability, which has strong consequences on the behaviour of these soils, to be clarified.

A thorough interpretation of mechanical laboratory tests results (on high quality undisturbed samples), has revealed the difference in behaviour between the stress levels lower and higher than the yield stresses, related to these soils' weak relic structure. Special emphasis has been made on the definition of the stress-strain behaviour of this soil, both volumetric (by means of oedometric and isotropic consolidation tests) and shearing (through triaxial tests, using different state and test conditions).

Both laboratory and *in situ* tests results have assisted in characterising deformability and strength on residual soils:

– *in situ* testing has special relevance in the evaluation of mechanical properties, although parametric correlations, originally developed for transported soils with similar physical indices, between in situ testing results, such as SPT, CPT, CH, PMT, SBPT and DMT, and deformation and strength mechanical properties, have to be adapted to residual soils.
– indicial and typological classifications based on in situ tests, such as CPT based charts, identify very realistically the cementation and fabrics, as well as the stiffness and strength reserve of the residual soil; correlations between q_c and N_{60} (SPT) give higher values than those found for transported soils of similar grain size distribution;
– the evaluation of ϕ' from $(N_1)_{60}$ gives values close to the ones obtained by back-analysis of loading tests and on triaxial tests on high quality block samples , although it does not reflect the presence of the significant effective cohesion contribution; on the other hand, the evaluation of the angle of shearing resistance through CPT leads to high values, because of the simultaneous sensitivity of q_c towards frictional and cohesive shear strength components; whenever possible, it is advisable to use triaxial tests on high quality samples as they identify most directly and conveniently the two components of strength; the cohesive intercept is recognised as a net sign of the cemented structure of these soils and its identification has deep consequences in design;
– the relations between G_0 and q_c presented higher ratios than those corresponding to transported soils, even with reference to highly overconsolidated or cemented soils;
– values of ϕ' and c' deduced from pressuremeter (PMT and SBPT) or dilatometer (DMT) tests seem to be largely influenced by the importance of the stress paths; for this reason values interpreted from expansion theories can be more closely related to extension stress paths failure envelopes; some specific correlations have been proposed between these tests parameters and design data;
– the execution of either some load tests under different plate/footing sizes or, on the other, a single one with complementary settlement measurement on the surrounding surface have been shown to be a preferable way to evaluate the effect of the stiffness variation with depth on the scale effect for shallow foundation settlement evaluation; in residual soils these effects are as much or more important than in transported soils, due the natural decrease in weathering with depth; some results were discussed of an experimental work including load tests on plates (0.30 m and 0.60 m in diameter) and a rigid footing (1.20 m in diameter) and the corresponding overall analysis in order to obtain the soil strength parameters and the load-settlement answer in non-dimensional terms; the integration of the results obtained from the in situ tests (penetration tests) into the analysis of the load tests results, allowed some of the stiffness variation laws that were deduced by the former way to be validated;
– the evaluation of the coefficient of earth pressure at rest, K_0, in weathered masses from granite profiles suffers from several testing limitations, that explains the insufficient database in residual soils; regional experience indicate that K_0 values are usually low for high weathering degrees (W_6–W_5 rock masses, ranging from 0.35 to 0.50), becoming higher in moderate weathering levels (W_4–W_3 masses, with K_0 values close to unity).

The integration of the information, in terms of stiffness and strength, obtained from the experimental data (in situ and lab), to assess methods used for the design of shallow foundations, and in this case, considering the behaviour of residual residual soils, has been used to interpret a large scale footing load test.

PREFERENCES, SYMBOLS AND UNITS (LIST OF ABBREVIATIONS)

B	smallest size(width) of a shallow foundation (footing)
c_α	coefficient of secondary consolidation ($c_{\alpha\varepsilon}$)
c'	effective cohesion
CID; CK_0D	Triaxal tests: consolidated isotropically or under K_0 and drained
CH	Cross-Hole test
CPT	Static Cone Penetrometer Test
d	density
D	diameter of a footing or plate
	unidimensional compression modulus
D_r	relative density (Index of Density)

DMT	Marchetti Flat Dilatometer Test
DP	Dynamic Probing: DPL – light; DPSH – super-heavy
e, e_R	void ratio, relative void ratio
E	deformability (Young) modulus
E_{el}	deformability modulus deduced from the initially linear reload branch in triaxial tests
E_D	deformability modulus deduced from the initially linear reload branch in triaxial
E_{pm}	Ménard pressuremeter modulus
E_{pmur}	Ménard pressuremeter unload-reload modulus
$E_{sn\%}$	secant deformability modulus for deviatoric stresses of n% of the failure load
E_{ti}	initial tangent deformability modulus deduced by hyperbolic modeling
E_{ur}	deformability modulus taken between vertices of an intermediate unload-reload cycle
E_0	small strain (maximum) deformability (Young's) modulus
G	shear modulus
$G_0 = G_{max}$	small strain (maximum) shear modulus
I	shape and geometry factors for settlement solutions, considering homogeneous space
k	coefficient of permeability
K	stiffness (hyperbolic) parameter
K_0	coefficient of earth pressure at rest
n	stress dependency exponent
N_{SPT}	number of blows on SPT tests
N_{20}	number of blows on DPSH tests
N_{60}	N_{SPT} values on an equipment with 60% of the theoretical energy
OCR	overconsolidation ratio
p_a	atmospheric pressure (101.3 kPa)
p'	mean effective stress
p'_0	mean effective stress at rest
p^*_L	Ménard's net limit pressure
PLT	Plate Loading Test
PMT	Pre-bored Ménard Pressuremeter Test
q	deviatoric stress (σ_1–σ_3)
q_c	cone resistance on CPT
q_{c1}	normalized cone resistance on CPT
q_f	failure deviatoric stress
q_s (q_{ser})	average contact pressure on the base of a footing (under service)
q_{sj}	load level
q_{sref}	reference load level (for a specific criterion)
q_{ult}	ultimate load or bearing capacity load of a shallow foundation
s	settlement (vertical)
s_{ref}	reference settlement for a specific load or deformation level
S_r	degree of saturation
SBPT	Self Boring Pressuremeter Test
SPT	Standard Penetration Test
V_s	shear wave's velocity
X_d	degree of decomposition
W_i	class of weathering of rocks
w	moisture content
z	depth (from the ground surface or the base of the loading area)
β	lixiviation index
ε	strain (ε_a: axial; ε_v:volumetric)
γ	unit weight
ϕ'	angle of shearing resistance ("friction angle") in terms of effective stresses
ϕ'_{cv}	angle of shearing resistance in critical state
ϕ'_{cv}	peak angle of shearing resistance
ν	Poisson ratio
σ, σ'	total and effective stress
σ_c	consolidation or confinement stress
σ_f	failure stress

σ_h	horizontal stress
$\sigma_v = \sigma_z$	vertical stress
τ	shear stress
τ_f	failure shear stress
ψ	angle of dilatancy

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