PORTUGUESE EXPERIENCE ON GEOTECHNICAL CHARACTERIZATION OF RESIDUAL SOILS FROM GRANITE

EXPERIENCE PORTUGAISE A LA CHARACTERIZATION DES SOLS RESIDUELS DU GRANIT

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SYNOPSIS: Being the most common soils in the north of Portugal, residual soils from granite were object of a research program aimed to a better understanding of the following aspects: (i) heterogeneity of soil profiles and depth of weathering; (ii) particle size distribution and plasticity characteristics; (iii) bonded structuring and its influence on mechanical properties; (iv) compaction characteristics and classification; (v) at rest stress state; (vi) stress-deformation and shear strength parameters; (vii) sampling and laboratory testing techniques; (viii) correlations between in situ tests (including SPT, CPT, DP and plate loading tests) and laboratory test results (namely, triaxials). In this paper some of these results are presented and discussed.

INTRODUCTION

In northern region of Portugal, where the climate is temperate, granite rocks are the dominant geological formations. It is typical the occurrence of a weathered superficial zone with residual soils whose thickness in some cases reaches 20m or more. This happens mainly in the coastal zone where most urban centers are concentrated. A sound geotechnical knowledge of these soils is therefore of great interest.

Two institutions from this region - Faculty of Engineering of the University of Porto and University of Minho - have been involved since 1987 in a research program aimed at a better understanding of such soils behaviour. This paper presents some results obtained in that research project.

WEATHERING FACTORS, PROFILES AND PHYSICAL CHARACTERISTICS

Analysing data from 24 dams built on granite in the North and Centre of Portugal it was observed that the main weathering factors seem to be the discontinuity degree, that is, number, spacing, orientation and continuity of joints as well as the proximity of other tectonic accidents. On the other hand, weathered profile thickness, seems to be independent on average annual precipitation, attitude of the soil surface and parent rock composition. Depth of weathering ranges from 0 to 20 m with more common values of 5 to 9 m.

Particle size distribution, Atterberg limits and Los Angeles abrasion tests indicate that the composition and texture of parent rock do not greatly influence those values. That is, saprolites from weathering of either coarse, medium or fine grain granites lead to residual soils displaying good grading and low plasticity indices. They are usually classified as silty sands (SM) or in some cases clayey sands (SC).

Fig. 1 shows typical particle size distribution curves taken from over a hundred analyses, represented inside a spindle except for a few curves which are explicitly drawn. Fig. 2 represents the plasticity chart with over 200 results of Atterberg limits tests (about 30% were classified as Non Plastic and, obviously, are not represented).

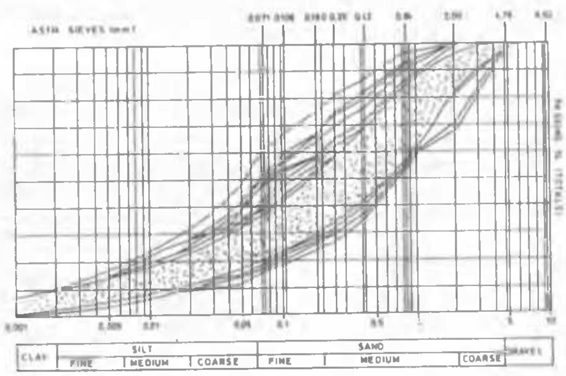

Fig. 1. Particle size distribution trends: the spindle contains over 100 curves

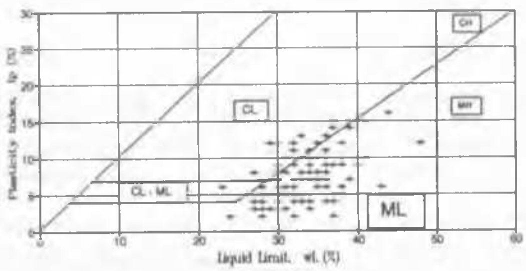

Fig. 2. Plasticity chart representation: 200 cases

The main classification of fines is ML, with some CL, which reflects a low plasticity behaviour. Typical values of w_l (25 - 40%) and I_p (2-13%) may reflect the influence of a high percentage of mica and feldspar which retain water in the internal planes of cleavage.

Table 1. Common natural physical parameters

$\gamma_s (kN/m^3)$	$w(\%)$	$S_r(\%)$	$\gamma_d(kN/m^3)$	e	$k(cm/s)$
25.7-26.5	15-25	80-100	15.0-18.5	0.40-0.70	10^{-4} -10^{-3}

Table 1 presents typical regional values of physical parameters. The relatively low values of total unit weight (typically between 17 to 19 kN/m^3) are associated to a flocculated structure (open continuous voids on a cemented, bonded structure) which results from water lixiviation processes of the parent rock.

The relatively high values of permeability (obtained on undisturbed samples) demonstrate the strong influence of these natural microstructures on creating preferential flow networks. On studying this microstructural factor a comparable study with remoulded samples was made. The latter were obtained by particle disaggregation and recompaction to original void and moisture conditions. Values of k drop to 10^{-6}-10^{-5} cm/s in the remoulded condition, probably due to creation of a more homogeneous distribution of the different grain sizes.

COMPACTION CHARACTERISTICS

Compaction characteristics obtained on different specimens from the same material using different energies are indicated in Table 2. The obtained compaction curves reveal, when compared with other soils with the same grain size distribution: (i) smaller values of $\gamma_{dmáx}$ in relation to w_{opt}; (ii) correlation between $1/\gamma_{dmax}$ and the logathmic of energy is almost linear as it is usual in similar well graded transported soils; (iii) the energetic linear correlation has, nevertheless, lower angular coefficient.

Table 2. Compaction energy characteristics and optimum values

Energy	Hammer weight P(Kg)	Falling height H(m)	Number of layers C	Blows per layer, N	Energy of compaction (Kgm)	w_{opt} (%)	$\gamma_{d\,max}$ (kN/m^3)
E1				12	27.3	19.9	15.7
E2	2.49	0.305	3	25	57.0	17.5	16.4
E3				55	125.3	15.2	17.2
E4	4.57	0.450	5	55	570.6	11.3	18.6

Experience with these soils has shown a fairly good behaviour as fill material namely for road construction, with moderately high values of C.B.R. towards the most common values for the same type of soil and energies (Novais Ferreira and Viana da Fonseca, 1988).

INITIAL STRESS STATE

The uncertainty of the at rest stress state in granite residual soils and its importance in geotechnical analysis led to the determination of K_0 values by using undisturbed large samples on triaxial chambers. Using Bishop's ring for radial deformation control, values of K_0 of 0.35 were obtained which are rather low for common concepts applicable to transported soils. However, these values are have been confirmed in other works, namely with self boring pressuremeters tests, and they are explained by the weathering theory proposed by Vaughan and Kwan (1984).

Moreover, local experience on supported excavations shows no tendency for substantial displacements spreading over a wide area around the cut as typical in some deposits with very high initial horizontal stresses.

STRESS-DEFORMATION CHARACTERISTICS

Confined and Isotropic Loading

Tests have been performed of unidimensional (oedometric) and isotropic (100mm diameter samples triaxial tests) consolidations apparatus in order to define volumetric pre and pos yield behaviour. The curves obtained from these tests show two clear trends as to what concerns the value of the compressibility index:

Pre-yield cemented zone $(\sigma'_c < \sigma'_{pc}) : c_r = 0.03 - 0.07$

Post-yield destructured zone $(\sigma'_c > \sigma'_{pc}) : c_c = 0.10 - 0.30$

In between it was found a meta-stable exclusively structural zone, typical of these cemented materials as referred by Vaughan (1988).

Triaxial Loading

Introduction

Recent reports (Vaughan, 1988) have shown that stress-deformation characteristics are far more affected by sampling and testing techniques than strength parameters. Therefore, it is very important to reduce disturbance on sampling by using larger samplers or blocks and to improve testing techniques by using, for instance, local axial strain measurement devices in triaxial specimens. On the other hand, some care has to be taken in the procedure of saturation of specimens since it appears that the conventional method, utilizing high back pressures, damages the natural cemented structure of the soil (Bressani and Vaughan, 1989).

Classical compression paths

A number of triaxial tests has been performed on samples obtained from one site inside the tunnel adjacent to the new railway bridge over Rio Douro in Vila Nova de Gaia. Sampling was undertaken in the vicinity of a given point in order to reduce difference between samples. These were tested under different conditions as following: (i) the samples were 38mm or 100mm in diameter; (ii) samples were tested either under drained conditions (with the natural water content which corresponds to a degree of saturation of 80-90%) or undrained (saturated by application of back pressures up to 300 kPa); (iii) isotropic consolidation was adopted in most cases, with pressures equal to 50kPa since this value represents common states of stress under shallow foundations.

Table 3 presents some results in terms of secant Young modulus at 50% of failure. Some trends can be extracted from those values:

(i) the sample size is of major influence as one can observe from the drop in the modulus for the smaller diameter; the pattern of stress-strain curves clearly reflects the high stiffness behaviour for larger samples, less affected by sampling;

Table 3. Triaxial testing on different conditions. A survey of the secant Young modulus ($E_{s50\%}$)

Number	Test	Diameter (mm)	$S_r(\%)$ [Back pressure, kPa]	σ'_c (kPa)	$E_{s50\%}$ (MPa)
1-3	CID	38	80-90	50	5.3
				100	8.1
				200	13.7
4	CID	100	80-90	50	10.5
5	CIU	38	100 [300]	50	5.2
6	CIU	100	100 [300]	50	12.2

(ii) it is not obvious from the tests that the process of saturation explicitly damages the soil structure as the values for secant Young modulus are reasonably similar in undrained conditions (saturated with high back pressures) and drained conditions (natural water content), which will represent the in situ modulus for common static loading.

The influence of suction was concluded to be of minor importance by determining the suction potential for natural moisture conditions (S_r in the range of 80 to 90%). From a practical point of view testing can therefore be performed without recourse to saturation by backpressuring.

Influence of the stress path

Figure 3 shows curves from CID triaxial tests on two different stress paths - axial compression (AC) and diametral extension (DE). Some conclusions may be extracted:

(i) in general, failure in stress paths with decreasing isotropic pressure (DE) is reached for lower strain levels than on the increasing isotropic stress paths (AC);

(ii) the pattern of stress-strain curves are distinct: initial stiffness on (AC) test is rather lower than in (DE) test and it remains lower until reaching 70% of failure; on the other hand, after this stress level drop in the Young modulus with stress is more pronounced in (DE) paths;

(iii) initial tangent Poisson ratios are higher in (DE) tests than in (AC) tests.

SHEAR STRENGTH CHARACTERISTICS

Table 4 presents strength parameters obtained from triaxial tests. SPT values for the same site and depth are also included when available. Some general trends can be deduced from these results:

(i) values of ϕ' are less variable than those of c', although in some situations the fact is possibly the outcoming of the assumption of a linear failure envelope;

(ii) the scatter of shear strength parameters reduces when larger samples are used because they are less dependent on soil heterogeneity and also the quality of sample is better;

(iii) friction angles obtained are relatively high, with lower bound limit of about 27-30°, corresponding to typical values of granular materials;

(iv) it is interesting to note that having tested materials with a large range of density (N_{SPT} from 7 to more than 60) the variation of friction angle values is rather smaller than that found for sandy transported soils with similar variation on N_{SPT} values; this can be related to the fact that in residual soils the factor "void index" is not so decisive in shear strength parameters as it is for sedimentary granular materials because the presence of open structures with some cementation between particles implies other concepts for indexing those materials mechanical behaviour (Rocha Filho et al, 1985);

(v) in general, differences in shear strength between more or less cemented levels will be reflected in values of c' which can rise from zero for remoulded materials up to values around 50 kPa for the lower range of effective consolidation stresses in natural conditions;

(vi) the soils exhibited peak behaviour and tendency to positive dilation (or to low to negative pore pressure generation in undrained tests), typical of overconsolidated or dense materials; this is in contradiction with their high void ratio and reflects their cemented structure.

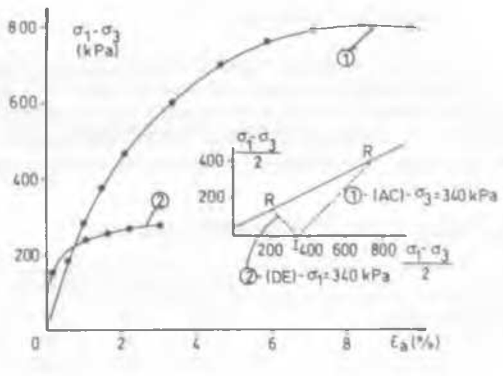

Fig. 3. Comparison of axial compression (AC) and diametral extension (DE) triaxial stress-strain curves

Table 4. Strength parameters for triaxial tests and some related SPT values

Site	Test	ϕ' (°)	c' (kPa)	SPT
S. João da Madeira	CIU Ax. Comp.	34-35	2-45	7-15
Oporto (City)	CID Ax. Comp. w_{nat}	28-31	17-34	12-17
		32	23	15-20
		37	5	>60
		30-33	40-52	--
	CID Diam. Ext.	31	18	15-20
Oporto (City)	CIU Ax. Comp.	33	24	10-16
		27	25	17
		37	32	22-32
Gaia Tunnel (1) (a)	(saturated) CIU Ax. Comp. ϕ=100	29-33	5-55	20-40
(2)	(saturated) CIU Ax. Comp. ϕ=38	31-38	6-43	20-40
(3)	(w_{nat}) CID Ax. Comp. ϕ=100	27-33	30-50	20-40
(4)	(w_{nat}) CID Ax. Comp. ϕ=38	35-39	9-27	20-40
(5)	ϕ=100 remoulded	32	7	--
(6)	CIU (sat.)	35	0	20-60
	CID (w_{nat})	30-40	35-40	20-60
	CID (w_{nat})	35	25	3-27
Matosinhos	CIU (ϕ38)	33	15	8-12
	CID (ϕ38)	31	13	8-12
	CID(ϕ100)	37	12	8-12
Braga	CIU (ϕ100)	32-41	0-3	--
	CID (ϕ100)	25-32	4-46	--

(a) Teixeira Duarte (1986).

SOME PARTICULAR CORRELATIONS USING "IN SITU" TEST PARAMETERS

Introduction

The study of residual soils is, as stated above, very affected by sampling and testing techniques so that it is of great interest the development of in situ testing methods for local direct determination of stiffness and strength parameters (pressuremeters, dilatometers, plate loading tests) or indirectly by establishing correlations with simpler methodologies (SPTs, CPTs, Dynamic Probing).

Relations Between N_{SPT} and N_{DP}

Table 5 includes technical data of the dynamic probing (DP) equipment employed in the sites indicated in Table 6. In this table the empirical values of the relation between the blow count of SPT and DP tests, N_{SPT}/N_{DP}, established for granite residual soils of northern Portugal, are compared with the theoretical (energetic) ones.

Table 5. Technical data of DP and SPT equiments

Equipment	DPL(1)	DPL(2)	DPSH(3)	SPT
Hammer weight, P(kgf)	15.0	10.0	63.5	63.5
Heigth of fall,H (cm)	40	50	75	76
Nominal area of cone, $S(cm^2)$	12.56	7.07	20.00	(A)20.42 (B)10.80
Penetration length, L(cm)	25	10	20	30
Specific energy per blow and per meter $E = PH/S\ L(kJ/m^3)$	192	710	1190	(A)787 (B)1490

(A) Projected area of whole shoe (2"O.D.)
(B) Projected area of whole annulus (2"O.D.-1"3/8 I.D.)
(1) - University of Minho; (2) Construções Técnicas;
(3) ISSMFE Technical Committee - 16.

Table 6. Theoretical and empirical values of N_{SPT}/N_{DP}

Source	Equipment (Table 5)	Empirical	Theoretical (A)	Theoretical (B)
510 cases	DPL(1)	0.21	0.24	0.13
6 holes $10 \le N_{SPT} \le 30$	DPL(2)	0.95	0.90	0.48
6 sites $15 \le N_{SPT} \le 50$	DPSH(3)	1.30-1.70	1.51	0.80
Matosinhos $8 \le N_{SPT} \le 22$	DPSH(3)	1.44-2.30	1.51	0.80

In general it seems to be clear that energetic relations taking into account the whole shoe of SPT fit fairly well with empirical correlations. The agreement between theoretical and empirical values supposing the whole shoe of SPT leads to the conclusion that SPT split spoon behaves like a "closed end cone" by the formation of a "strong plug".

Correlations Between $q_c(CPT)$ and N_{SPT}

In an attempt to determine ratios between CPT point resistance q_c, and the SPT blow count, N_{SPT}, some results were obtained in this research project which are represented in Table 7, together with others.

On the basis of the available results values of 0.35 to 0.50 MPa for q_c/N_{SPT} seem a reasonable proposal for the kind of residual soils apearing in the North of Portugal (lower values for finer soils). This value is quite typical of sandy soils and it is commonly accepted for transported soils.

Table 7. Correlations between q_c (CPT) and N_{SPT}

Source	q_c (CPT)/N_{SPT} (MPa)
Direct (200 cases)	0.35-0.60
Indirect (2 sites, N_{SPT} from DPSH(3))	0.30 for $q_c/f_s = 10-30$ 0.60 for $q_c/f_s = 30-60$
Direct (8 holes - Matosinhos)	0.40-0.60

Correlations Between "In Situ" Tests and Mechanical Parameters

Due to the typical heterogeneity of residual soils any attempt to correlate current in situ tests and strength and stiffness parameters or even bearing capacity (q_{ult}) should be exercised with great care. On this behalf one must concentrate experimental work on sites and depths very much localized so as to work for certain with the same soil. Complementarily, this emphasizes the recourse to the comparable experience for these testing and design parameters.

In general one can say that common strength correlations used for transported soils with similar grain size distribution are reasonably adapted to residual soils and can be even considered to be conservative.

On the deformation point of view some plate loading tests and results from laboratory tests with undisturbed samples revealed that correlation coefficients between q_c (CPT) and secant Young modulus (at 25% of failure) take the following equation:

$$E_d(\text{plates and CID triaxials}) = 1.2 - 1.4 q_c(CPT)$$

These values cannot be compared in absolute confidence with those presented in the literature for transported soils, mainly because it is not clear which level of stress the latter correspond to.

Some indications, however, that these soils can easily develop higher deformability due to creep effects, advise the adoption of these conservative correlation values.

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