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Routine and advanced analysis of mechanical in situ tests. Results on saprolitic soils from granites more or less mixed in Portugal

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ABSTRACT: This paper covers the more recent findings in the interpretation of different in-situ tests, such as SPT, CPT, PMT, SBPT, DMT and PLT to obtain geotechnical parameters of significant use in engineering practice. It concerns mainly shearing resistance properties and stiffness properties with special emphasis on the importance of stress and strain dependency. In this context some practical rules are presented for using parameters at two levels of design: routine and advanced levels. These practical rules concern transported soils (unaged and uncemented) are compared with those established in this paper for residual saprolitic soils from granite from different regions of Portugal.

It was noticed that the bonded structure and fabric of residual saprolitic soils from granite have a significant influence on their geomechanical behaviour. Consequently, the structural peculiarities of these residual soils influence the pattern of their non-linear constitutive behaviour. Deformability modulus derived from robust but relatively crude tests, such as SPT, CPT, DPT or even PMT, are compared with reference values taken from seismic survey (CH) and load tests, such as PLT. They can be situated on stress-strain levels defined from laboratory triaxial tests over high quality undisturbed samples. Several parametric correlations were established, which agree well with other correlations proposed for residual soils of the same nature. Significant differences are apparent between those correlations and the ones established for transported soils with identical grading curves, which may be explained by the weak bonded structure, inherited from the parent rock.

1 INTRODUCTION

The geotechnical site investigation is a function of the specific project and the associated risk. In general it takes into account the construction conditions and covers the following aspects (Hight and Higgins, 1995; Roberstson, 2001):

a. Geological regime: nature and sequence of the subsurface strata, stress history;
b. Groundwater regime: hydrogeological regime;
c. Soil and rock properties and behaviour: stress-strain-strength-creep-hydro properties and behaviour of the subsurface strata;
d. Geo-environmental regime: composition, distribution and flow of contaminants.

There are a variety of in-situ tests available to meet these objectives. A list of these major tests can be found in Lunne et al. (1997), where their applicability and usefulness for different type of ground conditions are described.

The best approach to use and choose the appropriate field tests is an integration approach involving structural engineer and geotechnical engineer. The approach should consider firstly the nature of the construction and the proposed methods of analysis and secondly the nature of the ground. Consequently the best in-situ tests for each given project and ground conditions will then be the ones which give the required information regarding the understanding of geological and geotechnical ground conditions and the relevant geotechnical properties and behaviour of the ground such as their constitutive laws to be used by the proposed methods of analysis used in the structural design. They should also give this information with an acceptable degree of accuracy at the lowest cost.

In this context it is interesting to classify the different types of ground behaviour used at different levels of structural analysis matching proper codes:

- Level No.1: routine calculation, assuming pseudo-elastic parameters for the ground (serviceability stiffness) obtained by routine analysis of test results;

- Level No.2: advanced calculation, assuming non-linear soil stiffness obtained by advanced analysis of tests results and;
- Level No.3: research calculation, using complex soil models obtained by analysing test results with complex constitutive laws of soil behaviour (hydro-stress-strain-strength-creep).

However, the use of these codes addresses a major practical difficulty which is related to the choice of the characteristic values to give to the relevant parameters of the constitutive laws. These values are usually obtained from laboratory tests where distribution of stresses and strains are homogeneous and boundary conditions are well defined. However, it is always difficult to obtain undisturbed samples; moreover, the selection of samples and their size can lead to uncertainty regarding the way they represent the soil. Consequently, in-situ tests could then become an alternative means to obtain these values. In any case it is always interesting to be able to compare the field test results with laboratory tests, and, if possible, combine both results. Unfortunately it must be pointed out that the drawback of most routine in-situ tests lies in the fact that the stress and strain distribution necessary for the identification of constitutive laws is unknown.

In this respect, it is also interesting to classify in-situ tests in three categories, regarding the method by which the stress-strain parameters are calculated:

- Category A: includes field measurements by seismic tests in which the small strain shear modulus G_0 is determined by using a sound theoretical basis;

- Category B: includes field tests such as pressuremeter tests, and plate load tests (PLT) which can yield deformation parameters using also a sound theoretical approach, but with a few or more assumptions or approximations. In this last context cone penetration test (CPT) and Marchetti dilatometer test (DMT) can also be included;

- Category C: includes field tests such as standard penetration test (SPT), CPT and DMT, for which the soil reaction cannot be easily modelled by a theory and deformation parameters are obtained by empirical correlations.

The importance of this classification is that only categories A and B can be expected to be used universally, while category C will apply only for the cases for which they were established and different correlations may be required for different soils (Atkinson and Sallfors, 1991).

This paper describes the most recent findings of tests of category B and C the results of which are suitable to characterise the stress-strain behaviour of soils, particularly stiffness, and that can be used for levels Nos.1 and 2 of design, mostly used by practising engineers. The tests of category A and the analysis involving level No.3 of design are covered in other papers (Stokoe et al., 2004, Yu, 2004). However, test results of category A will be used in this paper, since they are a fundamental parameter of the ground, considered as a benchmark value (Tatsuoka et al., 1997).

Emphasis is given to results obtained at experimental sites of residual saprolitic soils of the Centre (Guarda) and North of Portugal (Porto sites Nos.1and 2, and FEUP site). In fact, these soils originating from granite constitute the main geotechnical ambient for foundation design in most urban areas. Their cemented structure and fabric influence the engineering behaviour, particularly the stiffness, often estimated from in situ tests.

These experimental sites have been chosen in different regions of the Portuguese territory in order to establish fundamental correlations between simple parameters, such as cone-penetration resistances or pressuremeter data with deformability moduli (Viana da Fonseca, 1996, 1998, 2003, Viana da Fonseca et al. 1994, 1997, 1998, 2001, 2003, 2004, Duarte, 2002, Rodrigues, 2003). The use of seismic in situ tests for the evaluation of shear and compression wave velocities has enabled more precise reference values for stiffness. Some of these surveys included load tests on prototype footing or plates with different sizes, as well as laboratory tests on high quality samples, for the purpose of predicting foundation settlements using more or less complex models. The comparison between derived moduli is most relevant, in order to rely on the premises of design, mainly for service conditions.

Geotechnical characterization of typical granitic residual soils profiles from the metropolitan area of Porto based on extensive *in situ* testing allowed the discussion of some particularities for the terms used to derive geotechnical design parameters.

A first synthesis of two experimental sites (Porto sites Nos.1 and 2) was reported on Viana da Fonseca et al. (2001), a third survey (FEUP site) was included in Viana da Fonseca et al. (2004). Another carefully tested site (Guarda) will be considered herein, this one based on the work from Rodrigues (2003).

These soils were classified (by laboratory identification on undisturbed samples) as silty sands and sandy silts, more rarely as clayey siltes, being in agreement with DMT and CPT based classifications.

2 BACKGROUND

2.1 *Basic soil behaviour, parameters and properties*

Damage of civil engineering structures interests the pre-failure behaviour of the ground (Burland, 1989, Burland and Wroth, 1974). Presently there is a general consensus that the range of strains interesting the serviceability of structures is between 0,001% and 0,5% (Burland, 1989, Gomes Correia and Biarez, 1999, Biarez et al., 1999, Jardine, 1995, Simpson, 2001). Consequently ground behaviour in this deformation range (from small to medium strains) must be accurately characterised. Figure 1 summarises the main soil features with respect to strain level, including the positioning of in-situ tests in this context.

It is well recognised that soil exhibits an approximate elastic behaviour at very small and small strains and a non-linear behaviour at medium strains. This non-linear pre-failure behaviour complicates in-situ test interpretation and may conflict with simplifying assumptions made in the past. It is then crucial to define and identify the type of modulus that will be adopted. Figure 2 defines different modulus that can be associated with the pre-failure deformation of the soil.

It is obvious, that any correlation with a soil modulus must specify the type of equipment used and also the level of strain. The direct use of this modulus in practical applications will only be suitable if it is defined for the magnitude of strain that the soil shall exhibit at the site under working conditions (serviceability modulus or stiffness).

In routine design (level N° 1) the serviceability state can also be reached indirectly by using a global safety factor to the stresses obtained by ultimate limit state analysis. Furthermore, in advanced design (level N° 2) the commercial FEM geotechnical codes use simple elastoplastic models, needing information about post-peak shearing resistance. This, involves large and very large strains, for which range it is also recognised the post peak differences in strength due to the influence of dilatancy (Fig. 3).

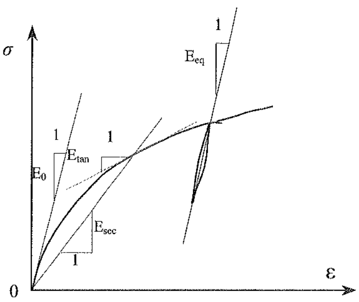


Figure 2. Schematic definition of the different moduli on a stress-strain curve

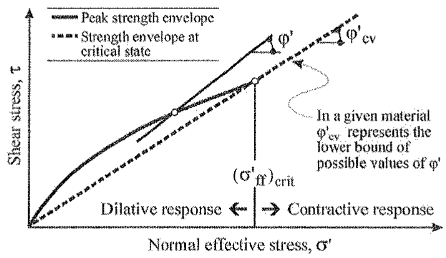


Figure 3. Schematic definition of different angle of shearing resistance (Randolph et al., 2004)

As a consequence it is necessary to identify different frictions angles: (1) peak angle of shearing resistance, ϕ'_p, (2) angle of shearing resistance at critical state, ϕ'_{cv}, and (3) angle of shearing at residual state, ϕ'_r.

Following the same attitude as for moduli, any correlation to be established with an angle of shearing resistance must specify which one is used. Present updated and detailed information about the pressure dependency of the angle of shearing resistance useful for foundation design can be found in Randolph et al. (2004).

<table>
<tr><td>Shear strain (%)</td><td colspan="6">0,001   0,01   0,1   1   10</td></tr>
<tr><td>Strain level</td><td>Small</td><td colspan="3">Medium</td><td colspan="2">Large - failure</td></tr>
<tr><td rowspan="3">Soil behaviour</td><td>Quasi-elastic</td><td colspan="5">Elastoplastic - failure</td></tr>
<tr><td colspan="5"></td><td>Critical state, residual state</td></tr>
<tr><td colspan="3">No dilatancy</td><td colspan="3">Dilatancy</td></tr>
<tr><td>Analysis for design</td><td colspan="4">Deformation analysis</td><td colspan="2">Ultimate state: bearing capacity and stability analisys</td></tr>
<tr><td>Structures (serviceability state)</td><td colspan="2">Roads, airports, rail track</td><td colspan="3">Foundations
Tunnels
Retaining walls</td><td></td></tr>
<tr><td>In-situ test operation</td><td colspan="2">Seismic tests: CH, DH, SASW, SCPT, SDMT</td><td colspan="3">LPT, SBPT (unload-reload)</td><td>SPT, CPT, PMT, SBPT, DMT</td></tr>
</table>

Figure 1. Important aspects related with soil strain level

Table 1. Values of Q and ϕ'_{cv} for different uncemented sands (Randolph et al., 2004)

SAND	MINERALOGY		Q	ϕ'_{cv} (°)	REFERENCE
TICINO	Siliceous (**)		10,8	33,5	Jamiolkowski et al. (2003)
TOYOURA	Quartz		9,8	32	
HOKKSUND	Siliceous (**)		9,2	34	
MOL	Quartz		10	31,6	Yoon (1991)
OTTAWA	Quartz	Fines 0%	9,8	30	Salgado et al. (2000)
		Fines 5%	10,9	32,3	
		Fines 10%	10,8	32,9	
		Fines 15%	10	33,1	
		Fines 20%	9,9	33,5	
ANTWERPIAN	Quartz and Glauconite		7,8 to 8,5	31,5	Yoon (1991)
KENYA	Calcareous		8,5	40,2	Jamiolkowski et al. (2003)
QUIOU	Calcareous		7,5	41,7	

In practical terms ϕ'_{cv} and ϕ'_p can be related by the empirical strength-dilatancy relationship proposed originally by Bolton (1986):

$$\phi'_p - \phi'_{cv} = m\{D_R[Q - \ln(\sigma'_{mf})]\} - R \;;\phi'_p \geq \phi'_{cv} \tag{1}$$

where: m is a coefficient respectively equal to 3 and 5 for axisymmetric and plane strain conditions ; R is a term, in first approximation function of (ϕ'_{cv} - ϕ'_{μ}), for sands $\cong$ 1; Q is a logarithmic function of grains compressive strength (Table 1) and σ'_{mf} the mean effective stress at failure. The importance of the relative density index becomes evident. Its value can only be obtained by field tests and this will be addressed in this paper.

2.2 *G_0 a benchmark value obtained by seismic tests*

The small strain shear modulus G_0 is the initial stiffness of the stress-strain curve for a given soil. In isotropic conditions, is related with the Young's modulus E_0 (Fig. 2) by $G_0=E_0/[2(1+\nu)]$.

This modulus, if properly normalised with respect to void ratio and effective stress, is in practical terms independent of the type of loading, number of loading cycles, strain rate and stress/strain history. It is then a fundamental parameter of the ground, considered as a benchmark value, which reveals the true elastic behaviour of the ground.

The first expression relating small strain shear modulus with void ratio and effective stress was derived in the early sixties by Hardin and Richard (1963) from field and laboratory tests on granular materials. This expression has been modified by different authors to accommodate clays and non isotropic conditions too. In a simplified form the expression can be written as:

$$G_0 = S \cdot p_a^{1-n} \cdot F(e) \cdot p'^n \tag{2}$$

where p_a is a reference stress, generally assumed equal to 100 kPa;

p' is the effective mean stress;

S and n are experimental constants and

$F(e)$ the void ratio function generally adopted as:

$$F(e) = \frac{(C-e)^2}{1+e} \tag{3}$$

where C is a constant, function of the shape and nature of grains.

More recently, based on results of six soft clays, Jamiolkowski et al. (1995) proposed the following equation:

$$F(e) = e^{-x} \tag{4}$$

In the following when the reference pressure (p_a) will be not used in formulae, then the value of S will expressed in pressure unities.

In the field the small strain shear modulus can be obtained by seismic tests (category A) which have in the last years opened new perspectives for the interpretation and use of their results in geotechnical engineering. This has been a consequence of the improvement of the testing equipment, signal processing and interpretation. On top of the evaluation of the small strain shear modulus, it is now well established that some more relevant information can be obtained by this category of tests:

1. evaluation of anisotropy by using polarised shear waves;
2. estimation of k_0;
3. evaluation of material damping;
4. evaluation of modulus degradation curve with strain;
5. evaluation of undrained behaviour and of the susceptibility of in situ materials to static or cyclic liquefaction.

When interpreting the results of in situ seismic tests it should be kept in mind that they are influenced by aging. This can explain why velocity of body waves of natural deposits of some age differ from that of same soil reconstituted in laboratory with the same state of effective stress and void ratio. Jamiolkowski et al. (1995) quantified the influence of aging on G_0 (see Table 2) by means of the following empirical formula (Anderson and Stokoe, 1978; Mesri, 1987):

$$G_0(t) = G_0(t_p)\left[1 + N_G \cdot \log\left(\frac{t}{t_p}\right)\right] \tag{5}$$

where $G_0(t)$ is the small shear modulus;
$G_0(t_p)$ as above at $t = t_p$, t is any generic time larger that t_p, t_p is the time to the end of primary consolidation and
N_G is a dimensionless parameter indicating the rate of increment of G_0 per log cycle of time (see Table 2).

Table 2. NG values to quantify aging in small shear modulus (Jamiolkowski et al. 1995)

Soil	d_{50} (mm)	PI (%)	N_G (%)	Notes
Ticino sand	0,54	-	1,2	Predominantly silica
Hokksund	0,45	-	1,1	Predominantly silica
Messina sand and gravel	2,10	-	2,2 to 3,5	Predominantly silica
Messina sandy gravel	4,00	-	2,2 to 3,5	Predominantly silica
Glauconite sand	0,22	-	3,9	50% quartzo, 50% glauconite
Quiou sand	0,71	-	5,3	Carbonatic
Kenya sand	0,13	-	12	Carbonatic
Pisa clay		23 to 46	13 to 19	
Avezzano silty clay		10 to 30	7 to 11	
Taranto clay		35 to 40	16	

Consequently comparison between in situ and laboratory measured values of velocity of seismic waves offers insight into the quality of the undisturbed samples. As already referred this type of tests will be considered in another lecture and will be not developed in this paper.

For a natural alluvial sands, aged and cemented, Ishihara (1986) proposed the following empirical equation to estimate G_0:

$$\frac{G_0(\text{MPa})}{F(e)} = [3.16 \text{ to } 5.72] \cdot \left[p'_0(\text{MPa}) \cdot 10^3\right]^{0.4} \tag{6}$$

where:

$$F(e) = \frac{(2.17 - e)^2}{1 + e} \tag{7}$$

Viana da Fonseca (1996) from results of cross-hole tests in a saprolitic soil of granite in Portugal (Porto) showed a very small stress dependency of the small strain shear modulus:

$$\frac{G_0(\text{MPa})}{F(e)} = 110 \cdot \left[p'_0(\text{kPa})\right]^{0.02} \tag{8}$$

where the void ratio function $F(e)$ was calculated using equation (7) based on results obtained on undisturbed samples recovered at the experimental site.

These results show that the constant value of the small strain shear modulus expression is much higher for these residual soils where S=110 kPa (eq. 2 without p_a) than for sandy transported soils where S=3.2 to 5.7, while the exponent n, reflecting the influence of the mean effective stress, is substantially lower. These different values of power n could be a consequence of different types of binding between grains (or glue) affecting the Hertz type of behaviour existing in particulate materials (Biarez et al., 1999).

More recent data for a Porto silty sand, Viana da Fonseca at el. (2004) found different constants, as illustrated in equation (9). This may result from the fact that the weathering conditions of the investigated soils are different. The comparison of these trends is presented in Figure 4.

$$\frac{G_0(\text{MPa})}{F(e)} = 65 \cdot \left[p'_0(\text{kPa})\right]^{0.07} \tag{9}$$

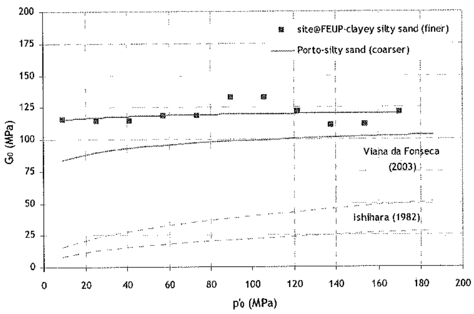


Figure 4. Comparison between observed and reference proposals of G_0 variation with effective stresses

This same analysis was made for Guarda's soils, another site in Portugal at Guarda, considering the void ratio that corresponds to the mean effective stress at that depth (Fig. 5). The two extreme values of the parameter S = 7,9 MPa and 14,3 MPa proposed by Ishihara (1982) were used in order to frame the results.

The results show that:

1. The magnitude of S and n parameters (eq. 2) reflects the reference value of the shear modulus to a certain degree. For the saprolitic granite under study, these values almost coincide with those found for the Porto granite by Viana da Fonseca (1996). These are both a great deal higher, however, than those indicated by Ishihara for the case of sedimentary materials. This express that the interparticular bonds present in the structured materials of residual origin have a predominant role in defining stiffness.
2. The value of parameter n exhibits significant differences in the case of the Guarda, as opposed to the Porto, saprolitic granite. These soils clearly show that a distinct dependence exists between the shear modulus and the in situ stress reflected in p'_0. In the case of the Guarda saprolite, the value of n is much closer to that proposed by Ishihara (1982) for aged cemented sands of sedimentary origin.

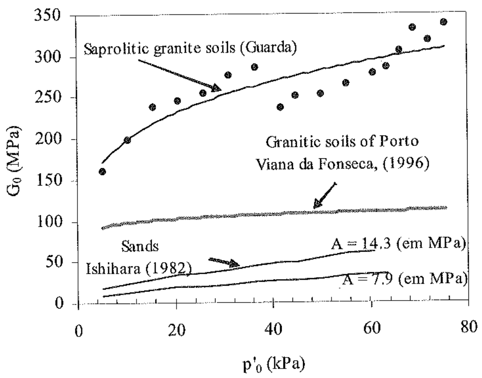


Figure 5. Relation between G_0 and p'_0 for the Guarda and Porto saprolitic granite and its conformity with the relation defined for granular sedimentary soils.

3 ROUTINE ANALYSIS OF MECHANICAL IN-SITU TESTS

3.1 *Elastic stiffness*

3.1.1 *SPT*

In many countries, as in Portugal, the Standard Penetration Test (SPT) is still a common in-situ test for geotechnical investigation.

A standardisation effort of SPT values has been done in relation with energy and depth influence, mainly for site liquefaction evaluation; this application of SPT values is not discussed here. Nowadays standardised SPT values for a energy ratio of 60% (N_{60}) is common.

Correlations between of SPT results and stiffness are very sensitive to different factors, while those relations between penetration parameters and small strain shear modulus (G_0) are somewhat independent of misleading factors, such as scale effects, non-linearity, etc (Jamiolkowski *et al.*, 1988).

From the many empirical correlations it is presented the one relating standardised SPT values with shear wave velocity, from which the small shear modulus is obtained (Seed et al., 1986):

$$Vs = 69 \cdot N_{60}^{0,17} \cdot Z^{0,2} \cdot F_A \cdot F_G \tag{10}$$

$$G_0 = \rho_t \cdot V_s^2 \tag{11}$$

where: Vs is the wave velocity (m/s), N_{60} is the number of blow/feet for a energy ratio of 60%, Z is the depth (m), F_G is a geological factor (clays=1; sands=1,086), F_A is the age factor (Holocene=1; Pleistocene=1,303), ρ is the total mass density.

Following Stroud's (1988) suggestion, a simple and very useful power law between G_0 and N_{60} is:

$$G_0(\text{MPa}) = C \cdot N_{60}{}^{n} \tag{12}$$

For the case of Porto granites the constant values were the following: $C = 63$; $n = 0.30$, for the first surveys (Viana da Fonseca, 2003), and, $C = 57$; $n = 0.20$, for the very last survey (Viana da Fonseca et al., 2004) – the former in a clear silty sand and the latter a clayey-sand with silt.

The variation of G_0 versus effective mean stress (p'_0) is very small when its variation versus other parameters, such as N_{60}, is analyzed. Correlations between G_0 and N_{60} for relevant values of p'_0 on shallow foundations are shown strongly underestimating elastic stiffness of these soils (Stroud, 1988).

Another methodology proposed by Schnaid (1999), is to establish the relation between G_0 and N_{60} by using normalised values as the following law:

$$\frac{\left(G_0 / p_a \right)}{N_{60}} = \alpha \cdot N_{60} \sqrt{\frac{p_a}{\sigma'_{v0}}} \tag{13}$$

in which, p_a – atmospheric pressure; α – adimensional value that reflects the dependence on interparticular bonds of the soil.

This will be displayed in a way such as Figure 6, where results from Brazilian saprolitic and lateritic soils, as well as those from Porto are included (Schnaid, 1999 & 2004).

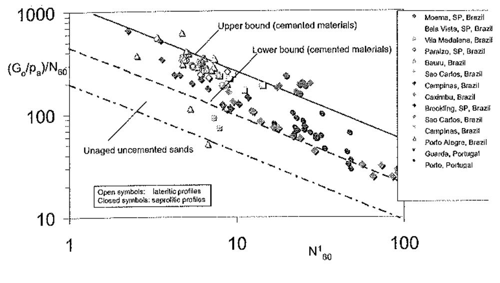


Figure 6. Initial stiffness normalized values predicted by the SPT test (Schnaid, 2004)

The results yielded by Porto (sites N°s. 1 and 2) and Guarda saprolitic granite soils are above those corresponding to the Brazilian saprolitic soils. They are the same order of magnitude as those found for the lateritic soils discussed by Schnaid (1999). This would explain the distinct regional peculiarities already suggested by other indicators, such as weathering characteristics, as revealed by various chemical and petrographic weathering indices (Viana da Fonseca, 1996, and Rodrigues, 2003), or even by the relationship established between tip resistance of CPT (q_c) and N. This value is also influenced by varying weathering conditions, tectonic history and parent rocks. Values obtained are clearly above those for the uncemented granular soils. The effect that interparticular bonds have on the behaviour of residual soils is thus confirmed. These bonds generate normalised stiffness values that are considerably higher than those for destructured soils with similar grading characteristics, void ratio and stresses.

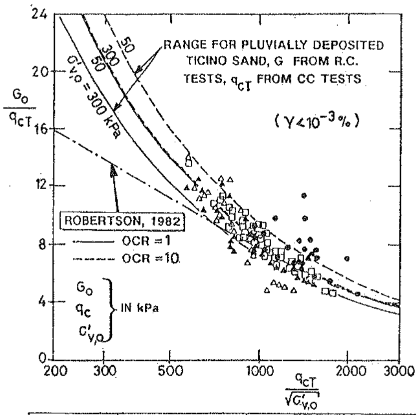
3.1.2 *CPT*

For a number of years engineers have attempted to correlate the cone resistance with different deformation moduli in order to predict settlements of structures. Most methods involve the estimation of some parameter linked to a certain calculation method. The bases of these equations are empirical and they all attempt to link observed settlements to cone resistance measured before construction. In some cases researchers have tried to correlate the cone resistance to deformation modules obtained in the laboratory. The deformation parameters of a soil are strongly dependent on stress history. Since the cone resistance, as well as the friction ratio are rather insensitive to stress history. Consequently, the deformation modulus cannot be expected to correlate well with the cone resistance, except for normally consolidated soils.

The only modulus which seems to be reasonably insensitive to stress history, as already mentioned, is the small strain shear modulus G_0. Consequently G_0 is the more appropriate to obtain a reliable correlation with cone resistance q_c. According to Jamiolkowski et al. (1988), this correlation can be established as a function of mean effective stress. Figure 7, based on field and calibration chamber data can be used to predict small shear modulus from CPT for uncemented sands.

Other correlations between q_c and G_0 for uncemented and unaged cohesionless soils, such as those given by Robertson (1991) and also Rix and Stokoe (1992) are represented in Figure 8. The last relationship relative to uncemented siliceous sands is expressed by the following equation:

$$\frac{G_0}{q_c} = 290.57\left[\frac{q_c}{(\sigma'_{vo}p_a)^{0.5}}\right]^{-0.75} \tag{14}$$



	SITE	SOIL	G_0 SOURCE	BOREHOLE
△	VIADANA	MEDIUM SAND	CROSS-HOLE	4017
▲	VIADANA	MEDIUM SAND	SEISMIC CONE	4017
□	S. PROSPERO	MEDIUM SAND	SEISMIC CONE	16 17
●	GIOIA TAURO	SAND WITH GRAVEL	CROSS-HOLE	209 219

DEPTH BELOW G.L. CONSIDERED: 5.5 TO 43.5 m

Figure 7. Normalized qc versus G_0 correlation for uncemented predominantly quartz sand (Jamiolkowski et al, 1988)

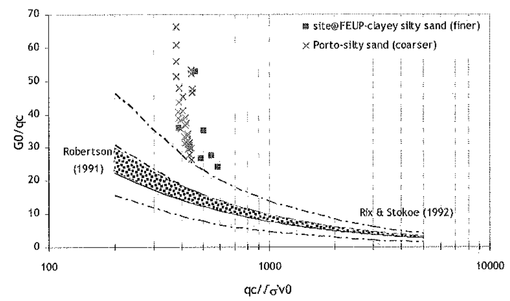


Figure 8. G_0/q_c versus $qc/\sqrt{\sigma'_{v0}}$ from in-situ tests at FEUP experimental site, compared with other regional data and with reference curves

However, the small shear strain modulus G_0 is better determined by geophysical methods. Information can now be obtained by the seismic cone SCPT, which can also include pore water pressure filter to become a SCPTu. Using this device, Mayne and Rix (1993) proposed the following empirical correlation between G_0 and cone penetration resistance based on a database gathering 31 clay sites results:

$$G_0 = \frac{99{,}5(p_a)^{0{,}305}(q_c)^{0{,}695}}{e_0^{1.130}} \tag{15}$$

More recently Jamiolkowski (2004), for sand and gravel of Pleistocene age at Messina straits obtained the following empirical equation:

$$\frac{G_0}{q_c} = 144.04\left[\frac{q_c}{(\sigma'_{vo}p_a)^{0.5}}\right]^{-0.631} \tag{16}$$

It must be pointed out that these purely empirical correlations, should only be applied to sites similar from those considered in the original database.

In fact, for cemented materials these correlations follow a completely different trend as is shown in Figure 8. In these materials results of CPT denote an approximately linear increase of q_c with σ'_{v0} (and depth), as shown in Figure 9. Robertson's (1990) classification 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.

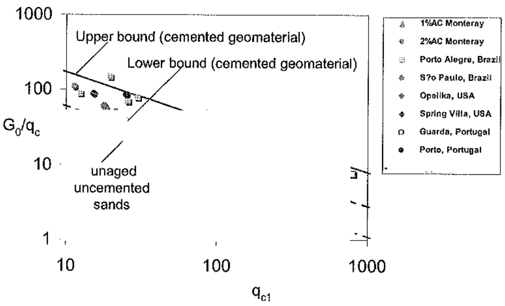


Figure 9. Results of CPT and CH over depth on Porto silty sand

The results of Figure 8 for the Porto saprolitic soil (silty sand), were obtained by means of CPT (q_c values) and cross hole (G_0 values) (Viana da Fonseca et al., 1998 and 2004).

Another way of doing this representation is by means of the proposals on Schnaid (1999), as shown in Figure 10. Again, the results yielded by Porto sites N°s 1 and 2 and Guarda saprolitic granite soils are above those corresponding to the Brazilian saprolitic soils.

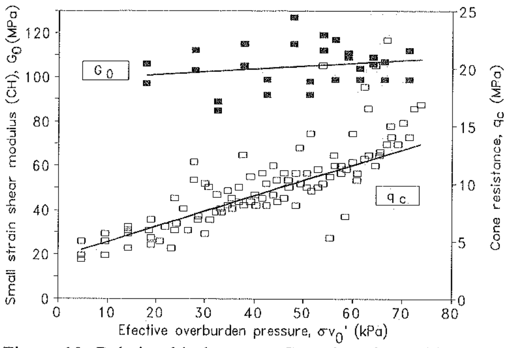


Figure 10. Relationship between G_0 and q_c for residual soils (Schnaid, 1999).

3.1.3 *PMT/SBPT*

Many kinds of pressuremeters probes are currently in use (Briaud, 1992, Clarke, 1995). Their differences are mostly related to the way they are inserted into the ground: predrilled hole (PMT), self-bored (SBPT), pushed-in (CPMT). It is obvious that the SBPT is the one that probe insertion causes a limited soil disturbance, contrary to the other types that cause an unavoidable stress relief. Consequently, the SBPT is the only one that can allow the measurement of the geostatic total horizontal stress σ_{h0}. It also offers a better interpretation of test results from small to large strains levels. Jamiolkowski and Manassero (1995) summarized the different geotechnical parameters that can be obtained by the three types of pressuremeters.

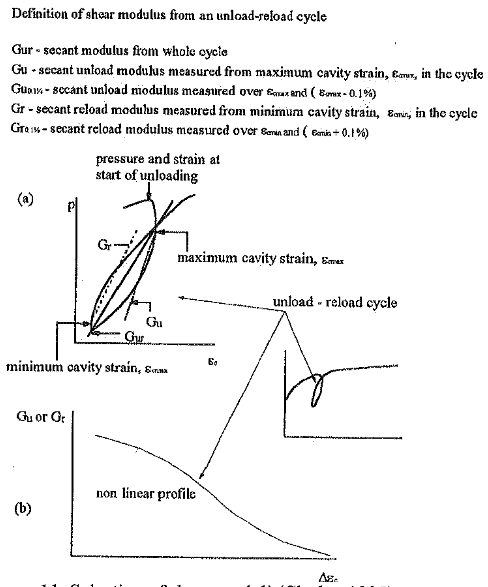


Figure 11. Selection of shear moduli (Clarke, 1995)

In Figure 11 are represented the different modulus that can be obtained by the SBPT.

Theoretically, the initial slope of a SBPT yields the G_0 value. However, in practice there is still some disturbance (Wroth, 1982) and, therefore, the modulus must be taken from an unload-reload cycle (G_{ur}). For very overconsolidated soils and cemented geomaterials it could be assumed that G_{ur}=G_0 if the strain of one cycle is less than 0,01%.

The use of G_{ur} in practice can be done by two approaches:

- To link G_{ur} to G_0 using a determined stress-strain relationship (Bellotti et al., 1989; Ghionna et al., 1994);
- To compare G_{ur} values to the degradation modulus curve G/G_0 versus shear strain - γ from laboratory, taking into account the average values of shear strain and mean plane effective stress associated with the soil around the expanded cavity (Bellotti et al., 1989).

The PMT is not appropriated to obtain directly G_0 because of the unavoidable disturbance during predrilling.

The SCPMT obtained by incorporating velocity geophones to a CPMT can directly measure G_0. As emphasized by Mayne (2001) new directions for enhanced geotechnical site characterization might optimize the amounts and types of data recorded. In this context the seismic piezocone pressuremeter (SPCPMT) seems to be an interesting tool (Fig. 12).

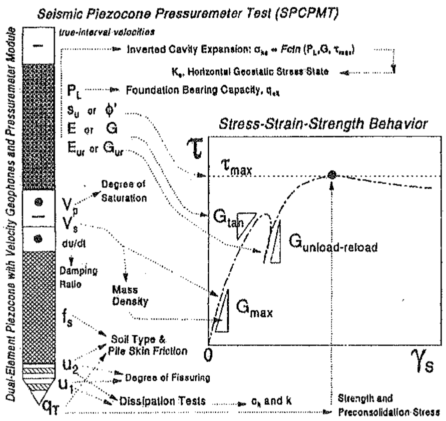


Figure 12. SPCPMT and recorded data availability (Mayne, 2001)

3.1.4 *DMT*

It is not the conventional dilatometer modulus E_D modulus which yields the best correlation between small strain shear modulus and dilatometer test results but the horizontal stress index (K_D), as pointed out by Marchetti (1997). Recently Tanaka and Tanaka (1998) for three sand sites found that G_0/E_D decreases as K_D increases. They observed the following trend: G_0/E_D decreases from ≈ 7,5 at small K_D (1,5 – 2) to ≈ 2 for K_D > 5.

It is also possible to incorporate velocity geophones to a DMT equipment and directly measure G_0 (Mayne, 1999).

3.2 *Serviceability Stiffness*

3.2.1 *Factoring G_0*

For practical purposes is necessary to extrapolate the results of small strains to the range of strain of engineering significance, generally 0,001% to 0,5%. This need arises from the recognition that the displacements of well designed civil engineering structures are generally quite small and overpredicted when using soil parameters that are inferred from conventional soil tests in theoretical settlement solutions (Burland, 1989; Tatsuoka et al., 1997; Simpsom 2001; Jardine et al., 2001).

Naturally the case of using a settlement solution based on the measured parameter is not considered here (Menard, 1962, Schmertmann, 1970).

A modified hyperbola can be used as a simple means to reduce the small strain shear modulus to secant values of G at working strain levels, in terms of shear strain γ, or at working load levels, in terms of the mobilized strength (q/q_u).

The generalized form may be given, in terms of γ, as:

Jardine et al. (1986):

$$\frac{G_s}{p'} = A + B\cdot\cos\left\{\alpha\cdot\left[\log_{10}\left(\frac{\varepsilon_D}{\sqrt{3}\cdot C}\right)\right]^{\gamma}\right\} \quad (17)$$

where: A, B, C, α, γ are constants; p' is the mean effective normal stress $p' = (\sigma_1' + \sigma_2' + \sigma_3')/3$ and:

$$\varepsilon_D = \left\{(2/3)\cdot\left[(\varepsilon_1-\varepsilon_2)^2 + (\varepsilon_2-\varepsilon_3)^2 + (\varepsilon_3-\varepsilon_1)^2\right]\right\}^{1/2} \quad (18)$$

or Gomes Correia et al. (2001):

$$\frac{G_s}{G_0} = \frac{1}{\left[1 + a\cdot\left(\frac{\gamma}{\gamma_{0,7}}\right)\right]} \quad (19)$$

where γ is shear strain;
$\gamma_{0,7}$ is the shear strain for a stiffness degradation factor of G/G_0=0.7 and
a is a constant (a ≈ 0,385, for the database used).

This relationship between G/G_0 and $\gamma^*=\gamma/\gamma_{0,7}$ seems to be very promising as a reference stiffness degradation curve, since, for the range of shear strain tested, it seems scarcely affected by the kind of soils (temperate or tropical soils), plasticity index, confining pressure, degree of saturation and overconsolidation ratio (Fig. 13).

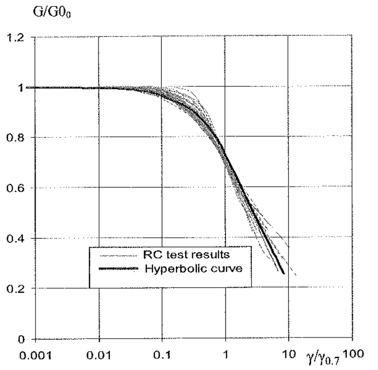


Figure 13. Relationship between normalized secant shear modulus and normalized shear strain ($\gamma/\gamma_{0.7}$) and hyperbolic fitting

Based on laboratory experimental results from 37 tests, by resonant column, of lateritic and saprolitic soils it was possible to establish the relationship presented in Figure 14 for lateritic and saprolitic Brazilian soils, allowing a practical use of these results.

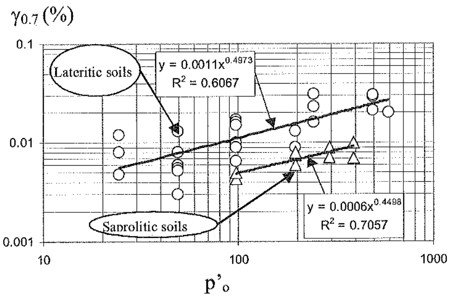


Figure 14. Relationships between $\gamma_{0.7}$ and p'_0 for lateritic and saprolitic soils

Puzrin and Burland (1998) proposed a more fundamental approach covering the full range of strains from small to large strains, through medium strains.

In terms of mobilized strength or stress level (q/q_u), the generalized form is (Fahey and Carter, 1993):

$$\frac{E}{E_0} = 1 - f\left(\frac{q}{q_{ult}}\right)^g \tag{20}$$

where f and g are fitting parameters. Values of f=1 and g=0,3 appear reasonable first order approximation for unstructured and uncemented geomaterials (Mayne, 2001).

The mobilized strength or stress level (q/q_u) can also be considered as an inverse factor of safety (FS), i.e. a stress level half of the ultimate corresponds to a FS = 2.

Using these functions factoring G_0 is possible to obtain a serviceability shear modulus, or serviceability stiffness, to be used in routine calculations to obtain settlements. As a very rough approach a factoring value of 0,5 can be used.

3.2.2 *SPT/CPT*

Viana da Fonseca and Almeida e Sousa (2001), for the Porto silty sand, using a crossed interpretation of footing and plate loading tests with the SPT values in the settlement influence zone, for a service level of q_s/q_{ult} ≅10-20%, obtained an average ratio between the serviceability secant Young's modulus and SPT values of:

$$E(\text{MPa})/N_{60} \cong 1 \tag{21}$$

This relationship is similar to the proposal of Stroud (1988) for normally consolidated soils, in identical stress levels.

The analysis of a large scale loading test (circular concrete footing 1.20m in diameter) and of two other plates of smaller diameter (0.30m and 0.60m), performed in the Porto silty sand, lead to the Young's moduli values presented in Table 3, for different loading stages. These results were obtained by back-analysis of the footing loading test (rigid footing), considering a linear elastic layer with constant modulus underlain by a rigid base at 6.0m depth.

Table 3. Secant Young's modulus, E_s, from loading test for different service criteria

Loading Tests	Service Criteria			
	q (s/B= 0.75%)	$q/q_{ult}$$^{(*)}$ (F_S=10)	q/q_{ult} (F_S = 4)	q/q_{ult} (F_S = 2)
Footing	17.3	20.7	16.0	11.0
Plate (0,6)	11.9	11.2	12.5	12.7
Plate (0,3)	6.7	6.9	5.9	5.7

(*) Corresponding to the allowable pressure for serviceability limit state design.

The intermediate stress level (FS = 4) corresponds approximately to the allowable pressure for residual soils, from Décourt´s (1992) criterion.

Correlations between q_c and Young's modulus, established for different stress-strain levels by triaxial tests (CID and CAD) with local strain measurements, confirmed the very strong influence of non-linearity on E/q_c ratios as well as a singular pattern of that variation when compared to proposals for transported soils (Viana da Fonseca et al., 1997).

The possibility of inferring design values for Young's modulus to predict the behavior of load tests, on plates and on a prototype footing, conducted in the vicinity of was these penetration tests was developed and thoroughly discussed elsewhere (summarized in Viana da Fonseca and Ferreira, 2002). The main conclusions drawn for the most common methods are as follows: the Burland and Burbidge (1985) equation based on SPT results led to an overestimation of the observed settlement by a factor of 2 to 3, while the application of the Schmertmann et al.'s (1978) method reproduced accurately the footing settlement for $\alpha = E/q_c$ values in the range of 4.0 to 4.5. Both methods identify this saprolitic soil in the global typology of cemented or overconsolidated granular soils.

3.2.3 *PMT/SBPT*

The routine analysis of PMT tests follows the method originally developed by Menard (1955). It gives design parameters directly obtained from the pressuremeter test curve (ASTM, 2004). Figure 15 shows the interpretation of the curve and Figure 16 exemplifies the procedure to obtain the pressuremeter modulus (E_m), based on the present ASTM (2004) standard. It must be pointed out that this modulus is related with the average stiffness exhibited by the ground associated with a determined strain level. Consequently the use of this value must be only applied in settlement formulae developed by Ménard (Ménard, 1963, 1965), as this is done in the French Code for foundation design (MELT, 1993, Gambin and Frank, 1995). Consequently Menard modulus must be considered as a test-specific design parameter.

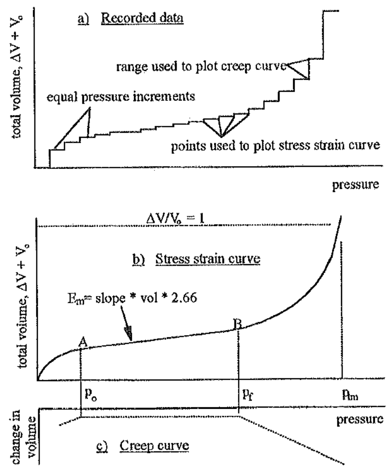


Figure 15. Interpretation of Menard test (PMT) according ASTM standard (Clarke and Gambin, 1998)

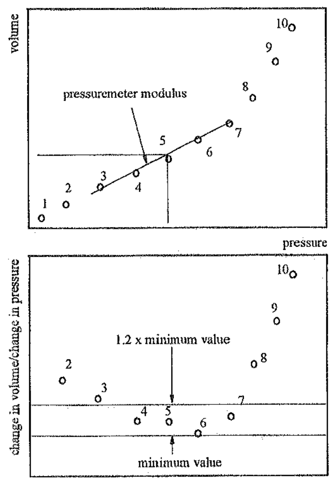


Figure 16. Selection of the pressure range to calculate E_m according ASTM standard (Clarke and Gambin, 1998)

Concerning SBPT, it is possible to analyse several moduli from SBPT results (Fig. 10): the initial shear modulus G_i (G_0 if enough precision is obtained) and the secant modulus from a unload-reload loop G_{ur}. This last modulus is judged to be more reliable and suitable to engineering practice, since G_i is strongly influenced by disturbance, even when small, and to the compliance of the measuring system (Fahey and Carter, 1993; Ghionna et al., 1994). However, even if this loops are suitable to measure shear stiffness, it is recognised the difficulty to associate this value to a strain level. According to Jamiolkowski and Manassero (1995) the value of G_{ur} measured in coarse grained soils represents the drained stiffness at intermediate strain level, between 1.10^{-4} to 1.10^{-3}, relatively insensitive to soil disturbance caused by probe insertion.

In practice, only the strain at the pressuremeter rubber cover surface is known, which means that the stiffness will be a little higher because stiffness is increasing further away from the pressuremeter. This assumption will allow obtaining degradation stiffness curve of the tested soil by varying the amplitude of the loop, which could be useful to compare with other test results.

Experimental in-situ work described by Viana da Fonseca (2003) revealed stiffness from reload-unload cycles of PMT (E_{pmur}) and SBPT tests in saprolitic granite soils apparently very different. In fact, for PMT it were found the following relations: $E_{pmur}/E_{pm} \cong 2$ and $E_0/E_{pm} \cong 18\text{-}20$, with E_0 determined on seismic survey (G_0 - CH), while for SBPT $G_0/G_{ru} \cong 2{,}6$ to $3{,}0$. It must be noticed that these last values are substantially lower than the ratio ($\cong 10$), reported by Tatsuoka & Shibuya (1992) on Japanese residual soils from granite. The non-linearity model of Akino - cited by the previous authors - developed for a high range of soil types, including residual soils, is expressed simply by:

$$E_{\sec} = E_0, \ \varepsilon \leq 10^{-4} \tag{22}$$

$$E_{\sec} = E_0 \cdot (\varepsilon / 10^{-4})^{-0.55}, \ \varepsilon \geq 10^{-4} \tag{23}$$

SBPT unload-reload modulus correspond to secant values for shear strain of about 6×10^{-4}, which agrees very well with the above indicated trends for this test (Viana da Fonseca, 2003).

It must be pointed that the comparison of results of the two types of tests can only be properly discussed if the mean effective stress during the cycle (p') is well estimated and the strain level of the cycle of each test reported. These aspects will be analyzed in item 4.

3.2.4 *DMT*

The modulus determined by the Marchetti's Flat Dilatometer (DMT), designated M_{DMT}, is the vertical confined (one dimensional) tangent modulus at σ'_{v0} and is said to be the same as E_{oed} ($=1/m_v$) obtained from an oedometer test in the same range of strains. This modulus can be converted in the Young's modulus (E) via the theory of elasticity. For ν=0,25-0,30 it is possible to write: $E \approx 0,8\ M_{DMT}$.

This empirical Marchetti's modulus is applied to predict settlements in sand and clays (Marchetti et al., 2001) and it was validated by different researchers (Schmertmann, 1986 in Marchetti et al. 2001 and Hayes, 1990, in Mayne, 2001).

Viana da Fonseca and Ferreira (2002) for characterization of the soil stiffness for shallow foundations settlement assessment, used correlations between the moduli E_{DMT} and $E_{s10\%}$ (secant modulus corresponding to 10% of peak shear strength). The following correlations were obtained:

$$G_0 / E_{DMT} \cong 16.7 - 16.3 \cdot \log_{10}(p_{0N}) \tag{24}$$

$$E_{s10\%} / E_{DMT} = 2.35 - 2.21 \cdot \log_{10}(p_{0N}) \tag{25}$$

These formulae are situated between those that are used for NC and OC transported soils.

3.3 *Angle of shear resistance*

In saturated geomaterials, drained and undrained conditions can prevail during in-situ testing. For penetration tests it is common to assume fully drained penetration in clean sands (drained conditions - ϕ') and for clays with very low permeability fully undrained conditions (s_u).

The undrained shear resistance (s_u) is greatly affected by several factors such as initial stress state, anisotropy, stress history, boundary conditions, strain rate, ...) and consequently it is generally normalized to the preconsolidation stress (σ'_p).

3.3.1 *SPT/CPT*

SPT can be used to predict the peak angle of shear resistance in granular soils when normalised to a reference energy (60%) and a stress-level of p_a= 100 kPa $(N_1)_{60}$, by:

$$(N_1)_{60} = \frac{N_{60}}{(\sigma'_v / p_a)^{0,5}} \tag{26}$$

Hatanaka and Uchida (1996) obtained the following equation, also corroborated by Mayne (2001) for residual silty sand in Atlanta and Georgia:

$$\phi'_p = [15,4(N_1)_{60}]^{0,5} + 20° \tag{27}$$

CPT is recognised primarily as a strength-measuring device (Houlsby, 2001).

Robertson and Campanela (1983) recommended for unaged, uncemented quartz sands the following correlation:

$$\phi'_p = \arctan[0,1 + 0,38 \cdot \log(q_c / \sigma'_{v0})] \tag{28}$$

An alternative equation considering the non linear normalization of qc with the stress level has been proposed by Kulhawy and Mayne (1990) in Marchetti et al. (2001):

$$\phi'_p = 17,6° + 11,0 \cdot \log(q_{c1}) \tag{29}$$

where q_{c1} is calculated by the following expression:

$$q_{c1} = \frac{q_c}{(\sigma'_v / p_a)^{0,5}} \tag{30}$$

A more general approach consist of estimating a secant friction angle. In fact, considering the non-linear shear resistance envelop defined in Figure 2 and equation 1, the secant friction angle can be estimated knowing the relative density D_r. This parameter can be estimated by means of equation 31 represented in Figure 17 (Jamiolkowski et al., 2003).

$$D_R = \frac{1}{C_2} \ell n \left(\frac{q_c}{C_o p_o'^{C_1}} \right) \tag{31}$$

where q_c and p'_o are both in kPa, and the various parameters are: C_o = 300 (dimensional); C_1 = 0.46; C_2 = 2.96.

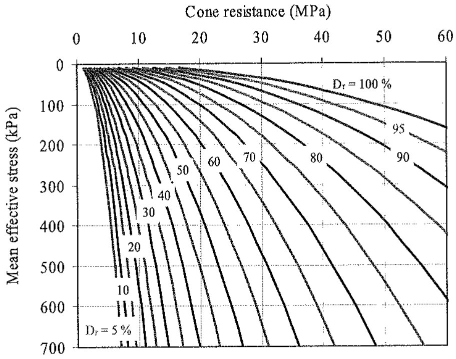


Figure 17. Relationship between cone resistance, relative density and mean effective stress for coarse-grained soils (Jamiolkowski et al. 2003)

In sands the pressure influences the peak and the dilatancy angles. However, as these angles are related (see Bolton, 1986 or Schanz and Vermeer, 1996), then as long as the angle of critical state is known, the friction angle can be deducted.

3.3.2 *PMT/SBPT*

Theoretically a peak and a post peak resistance can be obtained by pressuremeter tests. However, because the influence of disturbance during installation the peak resistance is usually ignored for PMT.

An usual prediction of undrained shearing resistance is obtained by the Ménard limit pressure - p_{lm} (Amar et al., 1975):

$$s_u = \frac{(p_{lm} - \sigma_h)}{5{,}5} \quad for \quad (p_{lm} - \sigma_h) < 300\,kPa \tag{32}$$

$$s_u = 25 + \frac{(p_{lm} - \sigma_h)}{10} \quad for \quad (p_{lm} - \sigma_h) > 300\,kPa \tag{32a}$$

where p_{lm} is the applied pressure required to double the cavity diameter and
p_0 is the estimated in-situ horizontal stress.

In the SBPT the following relationship can be used:

$$s_u = \frac{(p - \sigma_h)}{[1 + \ln(G/s_u) + \ln(\Delta V/V)]} \tag{33}$$

For drained conditions the angle of shearing resistance can also be estimated as follow:

$$\text{sen}\,\phi' = \frac{s}{[1 + (s-1)\text{sen}\,\phi'_{cv}]} \tag{34}$$

It is also possible to estimate both, drained and undrained shearing resistance by CPMT, but the method will be not presented here. For more details, see (Clarke and Gambin, 1998)

3.3.3 *DMT*

For undrained conditions the original correlation established by Marchetti (1980) is:

$$s_u = 0{,}22 \cdot \sigma'_{v0} \cdot (0{,}5 \cdot K_D)^{1{,}25} \tag{35}$$

This estimation of undrained shearing resistance seems according Marchetti et al (2001), to be quite accurate for design, at least for everyday practice.

Regarding the estimation of the drained angle of shearing resistance in sands two methods were proposed by Marchetti (1997). They both use the horizontal stress index k_D calculated by:

$$k_D = \frac{(p_0 - u_0)}{\sigma'_{v0}} \tag{36}$$

The use of a wedge plasticity solution relate I_D as a function of ϕ' and lateral stress state, including active pressure, at-rest k_0 (NC) value and passive pressure. Mayne (2001) found the following expression for the k_p case:

$$\phi' = 20° + \frac{1}{0{,}04 + 0{,}06/k_D} \tag{37}$$

This solution was later cross-correlated for CPT-DMT relationships by Campanella and Robertson (1991). Durgunoglu and Mitchel (1975), cited by Marchetti et al. (2001) presented a chart (Fig. 18) that allows the estimation of ϕ', in function of q_c, σ'_{vo} and k_0 (NC).

3.4 *Correlation between in-situ tests in residual soils*

The difficulties of sampling residual soils, which cause a number of problems for the characterization of stress-strain behaviour of soils through laboratory tests, make in situ tests very important tools in geotechnical practice. The most common tests are by far, the dynamic penetration tests, the classical SPT and in specific conditions dynamic probing (DPSH), but other more limited in penetration capacities (such as CPT and DMT) or more time consuming, such as PMT or PLT (plate load tests) are becoming more frequent as they give a more fundamental parametrical information. More recently a special attention is being made to seismic tests for the evaluation of initial shear modulus (G_0), regarded as a highly important benchmark parameter. Although in situ tests suffer serious limitations in terms of interpretation of their results, they nevertheless make a valuable contribution to geomechanical characterization.

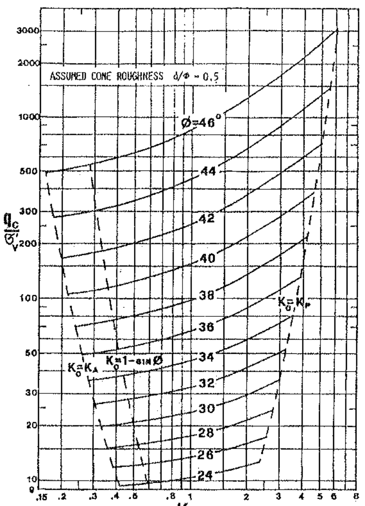


Figure 18. Estimation of ϕ', in function of q_c, σ'_{vo} and k_0 (NC), proposed by Durgunoglu and Mitchel (1975), in Marchetti et al. (2001)

It is interesting to examine if among the many correlations established between the results given by the various in situ tests some of them are applicable to residual soils, as a preference for evaluating strength and stiffness parameters, giving emphasis to the importance of the specific strain level associated with the deformability modulus derived from each of them.

3.4.1 *CPT – SPT correlations*

From their collection of data of different parent rocks of residual soils, Danzinger et al. (1998), concluded that correlations between CPT ad 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 (a pattern of soil type). It is assumed that from Brazilian data, there is a general trend of decreasing values of q_c/N_{SPT} with D_{50} and generally lower values than those expressed by Robertson and Campanella (1983) average line. Results from Porto granites, corroborate the very high sensitivity to the type of matrix, as it is expressed in Figure 18. In this figure, very recent data from the experimental FEUP site of the University of Porto is also included. Parts of these results were reported at this ISC'2 in the paper by Viana ad Fonseca et al. (2004). Results obtained are presented in Figure 19 including correlations proposed by other authors.

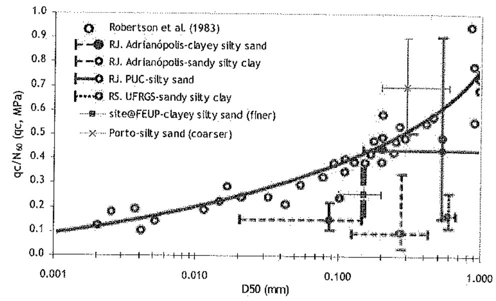


Figure 19. Ranges of q_c/N versus D_{50} on Brazilian residual soils, compared with the experimental site results (based on Danziger *et al*, 1998)

It is remarkable that data from this experimental site vary significantly with the dominant matrix of each soil. The results of the more silty sand Porto matrix shown in the previous data, exhibit a large contrast with the results of the more clayey soil in the last experimental site.

Corroborating this, Rodrigues & Lemos (2004) presented additional data obtained for the saprolitic granite soils from Guarda, a much coarser matrix, and plotted on the same graph (Figure 20).

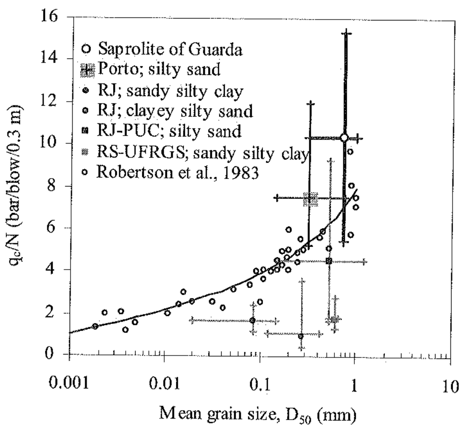


Figure 20. Values of q_c/N_{60} versus D_{50} for Guarda saprolitic granite and other residual soils.

The results obtained and presented on Figure 20 clearly show that, in the case of saprolitic soils from Guarda and the former Porto sites, q_c/N relations are conspicuously higher than those proposed for the granular sedimentary soils. This fact should be related to the greater sensitivity of the q_c parameter of the CPT test, than the value of N of the SPT test, concerning the cohesive part of the resistance, due to the existence of weak inter-particulate bonding and significant quartz coarse grains. It is indeed reasonable to accept that grain size distribution plays an important part in controlling stress-strain behaviour, since the coarser grain size of Guarda's saprolitic granite soils exhibits a higher q_c/N ratio than the saprolitic granite soils from Porto, whose grain size is finer. The Brazilian residual soils and the those from FEUP site have a q_c/N ratio that is lower than that predicted for sedimentary soils. Both theses soils have a similar mineralogical nature due to the original rock: the Brazilian rock being made of gneiss and sandstone, and the FEUP rock made of granite at the interface of gneiss and schist.

These findings lend further support to the idea that grain size properties do not in themselves explain the behaviour of the residual soils. They mean that other parameters must be incorporated into the analysis of the behaviour of these soils, namely, weathering indices, chemical and mineralogical ones.

An important aspect is the link between the drainage conditions during cone penetration and that expected in the design problem (Lunne et al., 1995). Takesue et al. (1995) showed this aspect for a volcanic soil, pointing out that the change in drainage, function of the penetration rate, has a larger effect on the sleeve friction than on the cone resistance. This is consistent with the fact that cone resistance is a total stress measurement contrarily to sleeve friction that is controlled by the effective stresses. The drainage also affects the relationship between CPT and SPT. In fact SPT is the summation of cutting shoe resistance and friction along the outside wall (and to a less extend along the inside wall) of the SPT sampler.

3.4.2 *PMT versus CPT/SPT correlations*

For the first two experimental sites on Porto granite saprolitic soils, there are some derived ratios between PMT and SPT or CPT parameters, which were reported in a paper by Viana da Fonseca et al. (2003). Theses correlations are included in the table 4.

Table 4. Ratios between SPT, CPT and PMT parameters

q_c/p_l*	f_s/p_l*	N_{60}/p_l* (MPa)	N_{60}/E_m (MPa)	E_m/p_l*	E_{mur}/E_m
14.3	0.390	14.6	1.4	10.6	1.4 - 1.9

fs is the friction sleeve of CPT; p_l* net limit pressure and the other symbols already defined.

3.4.3 *A synthesis of correlations obtained between in situ tests parameters and ratios between moduli*

In Table 5 comparative parameters between in situ tests are presented.

Table 5. Ratios obtained from in situ tests

q_d/q_c	0,75 – 1,25	E_m/p_l	12
N_{20}/q_c (MPa^{-1})	0,6 – 0,8	p_{0DMT}/p_{0m}	2 – 3
q_c/p_l*	4 – 6	P_{1DMT}/P_{fPMT}	≅ 1
f_s/p_l*	0,10 – 0,25	E_D/E_{PMT}	≅ 1,5

q_d is the dynamic tip resistance; N_{20} is the number of blows in 0,20m penetration of DPSHT; p_{0DMT} – Lift-off pressure of DMT; p_0 - Lift-off pressure of PMT (see Fig. 15); p_{lDMT} – Limit pressure of DMT; p_f – Creep pressure of PMT (see Fig. 15); p_l* net limit pressure

Ratios between distinct values of Young's moduli inferred from the investigations conducted have the obvious interest of fulfilling the needs of geotechnical designers to obtain data from different origins for each specific purpose.

Viana da Fonseca et al. (2003) reported some interesting correlations from the data available at the experimental sites:

– values of Young's moduli determined directly, with no empirical treatment, or even, no deriving assumptions;

– common constant ratios that are assumed to correlate SPT (DP) or CPT parameters with Young's modulus, comparing them with transported soils;

– relative values of moduli can be summarized in the way that is expressed in Table 6a, while some relations could be pointed out between in situ tests, as expressed in Table 6b.

– In what respects the relative position of the values deduced from the tri-axial tests on undisturbed samples, the data can be also summarized by some ratios presented in Table 6c.

Table 6a. Ratios between Young's modulus

$\frac{E_0(CH)}{E_{s1\%}(PLT)}$	$\frac{E_0(CH)}{E_{ur}(PLT)}$	$\frac{E_0(CH)}{E_m}$
≅ 8 – 15	≅ 2 - 3	≅ 20 - 30

Table 6b. Average ratios between Young's modulus and in situ "gross" tests

$\frac{E_0(CH)}{N_{60}(SPT)}$	$\frac{E_0(CH)}{q_c}$	$\frac{E_0(CH)}{q_d(DPL)}$	$\frac{E_0(CH)}{p_l}$
≅ 10 (MPa)	≅ 30	≅ 50	≅ 8

Table 6c. Ratios between Young's moduli obtained in tri-axial tests and in situ CH tests

$\frac{E_0(CH)}{E_0(BE)_{tx}}$	$\frac{E_0(CH)}{E_{el}(LI)_{tx}}$	$\frac{E_0(CH)}{E_{ur}(LI)_{tx}}$	$\frac{E_0(CH)}{E_{fi}(LI)_{tx}}$
≅ 2,0	≅ 2,4	≅ 3,1	≅ 4,5

Triaxial tests (tx): seismic waves velocities determined by bender elements (BE) and modulus in elastic loops (el) or between vertices on unload-reload cycles (ur), and secant to 10% of failure ($s_{10\%}$), using local instrumentation (LI).

3.4.4 *A synthesis of shearing resistance obtained between in situ tests*

The application of the proposal of Robertson and Campanella's (1983) to the first two surveys in the residual soils from Porto conducted to higher values of ϕ' than those derived both by the application of Décourt (1989) proposal based on SPT, or that one from DMT, following Marchetti (1997) correlation established for sandy soils. It should be noted that this correlation is assumed to underpredict ϕ', since the accepted value results from the lower limit of 3 curves based on Marchetti's assumptions, who considers K equal to K_{0nc} or to the square root of passive earth pressure coefficient (K_p). The reason of the discrepancy between SPT and CPT derived values of ϕ' (single resistance parameter that can be derived from tests that generated only "one" parameter) was discussed in Viana da Fonseca et al. (2003). This is a consequence of the high effective intercept on the vertical axis in the compression-shear domain, a pecularity of residual soils. This cannot be identified by a dynamic test, which reflects large strain strength, mostly ruled by the friction component, while the less destructive testing procedure of CPT is more sensitive to this low strain strength component.

This natural important deviations towards to the behaviour detected in transported soils modelled by the classical theories of Soils Mechanics are, to a great extent, due to a structural cementation inherited from the original rock mass and are, in terms of strength, essentially characterized by the existence of this effective cohesive intercept (c') and the development of a yielding behavior induced by the break of the cementation structure, independently from the failure corresponding to the plastific yield of the soil matrix component. The quantification of the cohesive resistance component (c') has been achieved mainly by triaxial tests and, less often, by back-analysis of load tests with plate or footing of different sizes. Getting undisturbed samples on these soils is extremely difficult, usually implying the partial or even complete loss of the cemented natural structure. Cruz et al. (2004) present an experimental conceptual approach, aiming at quantifying the effective cohesive component (c') of resistance by means of Marchetti's DMT. Since this test allows the determination of two basic parameters (p_0 and p_1), it is stated generating the possibility of evaluating both the angle of shear resistance and cohesive intercept. Assuming that K_D reflects the overall resistance of soil, it can be expected that either c' and ϕ' may affect this parameter. Then, if ϕ' from tri-axial testing is assumed, the corresponding K_D may be back-calculated. The difference between the two values of K_D (measured and back-calculated) will reveal the effective cohesive intercept. More detailed information can be found in Cruz et al. (1997).

Another issue that is also very pertinent for these residual soils is their particular sensitivity to sampling, since their behaviour is strongly controlled by the structure inherited from the parent rock. This issue was discussed in detail in other papers (Viana da Fonseca and Ferreira, 2000; 2002). It is also relevant to emphasizes the influence that the stress-path, mainly when the test is carried out in compression or in extension, has on the values of resistance parameters. As a clear illustration of this issue the derived values from in situ and laboratory tests results from the FEUP site are presented in Table 7 (extracted from Viana da Fonseca et al. 2004)

Table 7 Resistance parameters from in situ and laboratory tests

TESTS		ϕ' [°]	c' [kPa]
In situ	SPT	38	n/a
	CPT	37	n/a
	DMT	39	n/a
Laboratory	TX compression	45.8	4.5
	TX extension	28	12

4 ADVANCED ANALYSIS

The result of in situ test is either a penetration resistance or a relationship between some load (or stress) and the induced deflection (or strain); this result reflects the integrated and complex response of many soil elements around the instrument. It is then necessary to convert the test result by back analysis into soil parameters that at this level of analysis should be related with some soil model, which may be used in engineering design. However, this will require full understanding of the theories and models for which parameters are required (Atkinson and Sallfors, 1991). In this paper only simple soil models and structural models common in engineering practice are addressed.

4.1 *Simplified soil modelling – modulus degradation curve*

Elhakim and Mayne (2003) showed an approach to represent nonlinear stiffness soil behaviour based on CPT with seismic transducers, i.e. SCPT (Fig. 12). In this approach a modulus degradation graph is needed, like the one proposed before (see 3.2.1). In their work they choose Fahey and Carter (1993) degradation modulus (equation 20).

This same concept can be applied to PMT and SBPT with the incorporation of direct measurement of G_0.

SBPT offers also the possibility to assess the entire shear stress τ versus shear strain γ relationship in sands (drained) and in fine grained soils, undrained (Palmer, 1972; Manassero, 1989).

For undrained pressuremeter tests Wood (1990) obtained theoretical relationship between a non-linear pressuremeter test curve and a non-linear elastic stiffness-strain curve. This opened the possibility of using curve fitting methods to obtain non-linear undrained stiffness moduli from pressuremeter tests.

4.2 *Simple elastic-plastic soil model to derive test curves*

The common theories for the derivation of the test curves:

– cavity expansion curve, i.e. pressure versus strain curve in PMT, SBPT and DMT,

– load (or stress) versus displacement in plate load test (PLT))

mainly use simple elastic-plastic modelling of soil, considering stiffness as stress dependent.

Of course if a more realistic soil model is used, then it is obvious that a different test curve will be obtained.

It must be noticed that cavity expansion tests and PLT do not provide enough information to derive a unique solution of parameters of a simple elastic-plastic soil model. It is then useful to assess some of the parameters by other tests, mainly laboratory tests in order to narrow the range of possibilities. However, these parameters must be appropriated for the boundary conditions of the problem.

Among all the above-mentioned in-situ tests, the pressuremeter tests are the more appropriate for back analysis since they provide a complete pressure-strain curve for which elements of soil exhibit the same strain history at different radii distances from the surface. This last aspect is not the case for the PLT where each element of soil under the plate undergoes different strain history (Houlsby, 2001).

Gomes Correia et al. (2004) back-analysed PMT and PLT results on a silty sand (residual soil of Granite, close to Porto) using a simple model developed by PLAXIS and called HSM (hardening soil model) which assumes a non linear elastic response of the soil during loading and a isotropic hardening during unloading. The main features of this model are:

- Stress dependent stiffness.
- Plastic straining due to both primary deviatoric and compression loading.
- Elastic unloading and reloading.
- Failure according to the Mohr Coulomb model.

The stress-strain geotechnical parameters of the model are well known by professionals: stress dependent Young's modulus, Poisson's ratio, angle of shearing resistance at critical state, dilatancy angle and effective cohesion.

Some of the relevant results of this study are also pointed out here (Gomes Correia et al. 2004).

Using the previous modeling technique where the friction angle was derived from tri-axial tests (Fleureau et al., 2002), the following conclusions were drawn:

- The HSM in PLAXIS could be a good compromise to back-analyze PMT results, while for PLT the identification of the parameters of this non linear law from a load-settlement curve is more difficult without having other information. A possible solution to this is the measurement of deformations in depth under the plate with strain gauges, as it was proposed by Burland (1989) and Tatsuoka et al. (1989).

- Menard modulus (E_m) obtained in the routine analysis according to the ASTM (2004) standard (see Fig. 16) is associated to a strain level near 1 % (Biarez et al 1998; Gomes Correia et al. 2004). The secant modulus of the unload-reload cycle is around 2,2 times Ménard's modulus.

- The secant modulus of an unload-reload cycle of the plate load test is rather close to a strain level of 0,1 %. Furthermore, the unload-reload modulus of plate load test is about three times the unload-reload modulus of pressuremeter, as a consequence of the associated different strain levels, assuming that the representative stresses in the two tests are identical.

- These strain levels are in good agreement with the E-moduli values obtained for both types of tests.

- In the non linear elastic behaviour domain of the soil, the curve which expresses the variation of the applied pressure during a PLT versus the ratio settlement over diameter is close to the curve which shows the function of the vertical stress versus the vertical strain in a tri-axial test. In the numerical modeling with a power law equal to 0.5 (Hillier and Woods, 2001), the ratio between the relative plate deformations or "relative strains" δ/D and the tri-axial strains was about 0.5 (see Gomes Correia et al. 2004).

The routine interpretations of PMT and PLT led to very different values of modulus. Besides, the Ménard modulus is a tangent modulus (Fig. 2), in the sense that it is obtained by the slope of the pseudo-elastic zone of the pressuremeter curve, while the modulus obtained by the plate load test is generally a secant modulus. In addition it is obvious that these moduli will be modified if test procedure or interpretation is modified. This is a consequence of the non-linear material behaviour, where the modulus depends on the level of stress and strain, among others. The main point is to know in practice how to use correctly these values. In fact, modelling geotechnical structures is being more and more popular, and consequently the results of category B tests must be more and more used for the identification of the model parameters. It is evident that the correctness of this identification is a function of the model adopted. So, the appropriateness of the model must be carefully analysed and confirmed.

5 CONCLUSIONS

It is nowadays well established in the geotechnical community that soils exhibits non linear behaviour giving place to the definitions of different modules and angles of shearing resistance. This non-linear behaviour complicates in-situ test interpretation and may conflict with simplified assumptions made in the past. To clarify this, the following directions were presented in this paper:

– A direct use of moduli in practical application will be only suitable if it is defined for the magnitude of strain and stress that the soil shall exhibit at the site under working conditions. Otherwise it must be associated either with correction factors or by using design rules well calibrated by the real behaviour of structures, as is the case of Ménard's Modulus.

– Any correlation of in-situ test results should specify, furthermore the type of equipment and test procedure, the type of modulus or angle of shearing resistance;

– The small shear strain modulus (G_0), if normalised with respect to void ratio and effective stress, is in practical terms independent of the type of loading, number of loading cycles, strain rate and stress/strain history. Consequently it is the most appropriate parameter to establish correlations with in-situ tests (SPT, CPT, DMT). A great improvement will be the incorporation of seismic transducers with these equipments for routine site investigation work. It must be stressed that these correlations are only valid for the materials tested, but they are very useful for regional and country applications in order to create databanks. This will be of great help during design phase, since it will allow analysis to be developed, while specific tests results are not yet available.

– At routine design level, G_0 could be adapted to strain level of engineering significance (0,001 % to 0,5 %). Some rules are presented acting in terms of shearing strain or of the mobilized strength;

– The peak angle of shearing resistance seems to be well correlated with SPT and CPT results. It is also possible to correlate that value with the non linear secant angle knowing mainly the relative density;

– In advanced design level soil parameters of engineering significance to be used in soil modelling (non linear behaviour) can be obtained by two approaches:

- By using results of category C tests (SPT, CPT, DMT, PMT, SBPT) simultaneously with G_0, or G_{ur} (SBPT) with different strain amplitudes, to obtain a modulus degradation curve – simplified soil model;
- By back-analysing results of category C tests, stress-strain results of PMT, SBPT, or load-displacements results from PLT.

– The comparison of different modulus obtained by category C tests in the same soil can be done using some kind of analysis in order to situate the stress and strain associated to each modulus. In an experimental study on residual soils of granite it was obtained that E_m (PMT) according ASTM (2004) is associated to an strain level around 1%, while the unload-reload modulus of PLT according ASTM (1993) is close to 0,1%.

In this paper it also pointed out the peculiar behaviour of saprolitic granitic soils (aged and cemented) of some regions of Portugal putting in evidence the differences related with transported soils (unaged and uncemented). Several correlations between different tests of categories A, B and C, following the directions mentioned previously, are presented. They are very useful for day to day design practice and are being collected to update a knowledge based system already implemented covering a variety of geomaterials from rock to soils.

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