THE USE OF PRESSUREMETER TESTS FOR MODELING RESIDUAL SOILS GEOMECHANICS AND FOUNDATION BEHAVIOUR

L’UTILISATION DES ESSAIS PRESSIOMETRIQUES POUR LA MODÉLISATION DE LA GÉOMÉCANIQUE DES SOLS RÉSIDUELS ET LE COMPORTEMENT DES FONDATIONS

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“CHARACTERIZATION OF RESIDUAL SOILS”

INDEX

• Singularity of residual soils
• Background Characteristics
• In situ testing for mapping, classification and geomechanical characterization of residual soils
• Specific features in residual soils
• Conclusions
Singularity of residual soils

a) Young Residual Gneiss
b) Mature Residual Gneiss

- Complex profiles / macro-structural features;
- Heterogeneity;
- Structured soils;
- Can be in unsaturated condition.
Singularities of residual soils – complex profiles

Typical profiles of Brazilian residual soils

a) Metamorphic or Granitic Rocks  
Costal Range

b) Intrusive basaltic rocks  
Hinterland Plateau

c) Sedimentary (Sandstone) Rock  
Hinterland Plateau
### Singularities of residual soils – complex profiles

| Residual or transported soil (mature) | I – organic horizon |
| II – lateritic soil horizon |
| Residual soil (young) | III – saprolitic soil horizon |
| Transition from soil to rock | IV – saprolite horizon |
| V – highly weathered rock horizon |
| VI – weathered rock horizon |
| VII – sound rock |

**Typical profiles of Brazilian residual soils**

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Typical weathering profile of residual soils

- Grade VI residual soils
- Grade V completely decomposed
- Grade III moderately decomposed
- Grade I Fresh rock (Lower Zone)
Non-textbook residual soils- relic structures still prevail!

Behaviour of residual soils’ foundations, retaining walls, underground slopes..., is governed by macro and micro structural features: flocculated matrices with high voids and cemented such as relic jointing, bedding or slickensliding inherited from the parent rock.

The influence of relic structural features cannot be ignored in analyses of geotechnical works in residual soils; stiffness, strength, permeability and technical solutions are dictated by those singularities ...
Background Characteristics

- Index properties
- Mechanical properties: deformability and strength parameters (saturated soil)
- Mechanical properties in unsaturated condition
- Sampling a fount of uncertainty
INDEX PROPERTIES

Classification from in situ tests

Profile identification from distinct tests based classifications (Viana da Fonseca et al. 2006)

Robertson (CPT)  Eslami & Fellenius (CPT)  Marchetti (DMT)  Zhang & Tumay (CPTU)

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Classification from in Situ Tests – Cementation Effect

Classification charts with at least two independent measurements: $G_0$ and $q_c$

\[ G_0 = 800 \sqrt{q_c \sigma'_v p_a} \] upper bound : cemented
\[ G_0 = 280 \sqrt{q_c \sigma'_v p_a} \] lower bound : cemented
\[ G_0 = 110 \sqrt{q_c \sigma'_v p_a} \] lower bound : uncemented

\[ q_{c1} = \left( \frac{q_c}{p_a} \right) \sqrt{\frac{p_a}{\sigma'_v}} \]

### Notes:
- **Monterey**: 1% cemented
- **Monterey**: 2% cemented
- **Porto Alegre, Brazil**
- **Sao Paulo, Brazil**
- **Spring Villa, USA**
- **Opelika, USA**
- **Guarda, Portugal (PT)**
- **Matosinhos, Porto, PT**
- **CEFEUP, Porto, PT**

**Legend:**
- Blue diamonds: Monterey: 1% cemented
- Pink squares: Monterey: 2% cemented
- Blue triangles: Porto Alegre, Brazil
- Orange circles: Sao Paulo, Brazil
- Yellow diamonds: Spring Villa, USA
- Light blue crosses: Opelika, USA
- Orange triangles: Guarda, Portugal (PT)
- Red diamonds: Matosinhos, Porto, PT
- Dark brown circles: CEFEUP, Porto, PT

**Graph Notes:**
- Upper bound (cemented geomaterials)
- Lower bound (cemented geomaterials)
- Unaged uncemented sands
Or if SPT values (Schnaid et al. 2004)

\[
(N_1)_{60} = \left( \frac{N_{60}}{p_a} \right) \sqrt{\frac{p_a}{\sigma'_v}}
\]

versus \[
\frac{G_0}{N_{60}}
\]

![Graph showing relationship between \(G_0\) and \(N_{60}\)]

Relationship between \(G_0\) and \(N_{60}\)

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Mechanical Properties: Deformability Parameters (saturated soils)

Meta-stability – Virtual or apparent overconsolidation

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Relationship between Log $\sigma'_m$ and $e_o$

The cohesive-frictional nature of residual soils

Influence of mineralogy in strength of residuals soils from gneiss
(Sandroni, 1977, modified by Coutinho et al. 2004)

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Mechanical Properties: Strength Parameters (Sat. Soils)

**Anisotropy: Cohesive, frictional nature of strength and stress–strains answer**

Anisotropy of cohesive-frictional strength and stress-strain answer under different stress paths is only identified with high quality samples.

Highest evidence of cohesive tensile for compression path with decreasing of mean effective stress, in opposition to increasing of mean effective stress; notorious significance in the differences of geotechnical parameters obtained for in situ expansion tests (ex. pressuremeters) versus compressive tests (penetrating tools)!

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SAMPLING: a source of uncertainty

The measurement of deformability and shear strength of residual soils requires **samples of high quality**, and test specimens should be large enough to include large particles fabric elements.

Retractable triple-tube core-barrel (Mazier) and block sampling
Sampling quality as pre-request for good characterization

Reliable methodology for assessing sampling quality by the comparison of seismic wave velocities in situ and in laboratory specimens has been increasingly accepted as most promising, specially in structured soils, such as residual masses.

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Sampling methods

(a) Rough Carving of Sample
(b) Trimming of Sample
(c) Sealing of Void
(d) Separation from Parent Material

Block samples

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## Undisturbed tube samplers

<table>
<thead>
<tr>
<th>#</th>
<th>Sampler</th>
<th>Internal diameter [mm]</th>
<th>Liner</th>
<th>Sampling</th>
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<tbody>
<tr>
<td>S1</td>
<td>GMPV</td>
<td>72.0</td>
<td>grey PVC</td>
<td>dynamic</td>
</tr>
<tr>
<td>S2</td>
<td>ST85</td>
<td>75.0</td>
<td>PVC (often transparent)</td>
<td>dynamic</td>
</tr>
<tr>
<td>S3</td>
<td>NT81</td>
<td>74.0</td>
<td>coated steel (not stainless)</td>
<td>dynamic</td>
</tr>
<tr>
<td>S4</td>
<td>Mazier</td>
<td>*</td>
<td>blue PVC</td>
<td>rotary</td>
</tr>
<tr>
<td>S5</td>
<td>Osterberg</td>
<td>30</td>
<td>none</td>
<td>stationary</td>
</tr>
<tr>
<td>S6</td>
<td>Shelby</td>
<td>30</td>
<td>none</td>
<td>stationary</td>
</tr>
</tbody>
</table>

*sharp cutting edge ahead of the drill bit
Assessment of sampling quality

Comparison of seismic wave velocities

For the comparison between laboratory and in situ seismic wave velocities, the results were analysed at the estimated in situ stresses.

For a more consistent comparison, the shear wave velocities were normalised to the respective void ratio, as follows:

\[
V_S = \sqrt{\frac{G}{\rho}} = C \cdot \sqrt{F(e)} \cdot \sigma'_v^{n_a} \cdot \sigma'_h^{n_b}
\]

\[
F(e) = e^{-1.3}
\]

\[
V_S^* = \frac{V_S}{\sqrt{F(e)}}
\]
Assessment of sampling quality

Classification of sampling quality in Residual Soils

Ferreira et al. (2011)

Combined plot of the normalised laboratory $V_S$ values against the corresponding normalised in situ values (for the same mean effective stress):

The slope of each line provides an indication of the loss of shear wave velocity in relation to the field, which can be considered a measure of the sample disturbance.
In situ testing for mapping, classification and geomechanical characterization of residual soils

- Field Techniques
- Geophysical Survey – persecuting heterogeneity
- Geomechanical Characterization
- Overall fitting of SBP pressure-expansion curve
Field Techniques

Can be broadly divided into (Schnaid et al. 2004):

- **Non-destructive or semi-destructive tests**
  - geophysical in-situ methods, **pre-boring and self-boring pressuremeter (PMT, SBPT)** and plate loading (PLT)

- **Invasive, destructive tests**
  - SPT, CPT and DMT

The applicability and potential of existing techniques, implies a critical appraisal on how results can be compiled to obtain a ground model and appropriate geotechnical parameters.

- **Textbook soils vs Residual soils**
Geophysical Survey – persecuting heterogeneity of residual soils

Geophysical methods (seismic, electrical and electromagnetic) are determinant in near-surface site investigation and geotechnical characterization, in mapping the geological and geotechnical spatial variability of rock/soil mass formations.

Refraction tomography, electrical resistivity, radargram and other geophysical methods

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The parameters are obtained directly or indirectly from in situ tests, such as: density, at rest stress state (repos), resistance (to failure), deformability, permeability, etc.
Geomechanical Characterization – In-situ Tests

Correlations between their results - CPT vs. SPT

Ranges of $q_c/N$ versus $D_{50}$ on Brazilian and Portuguese residual soils, with experimental site results


A. Viana da Fonseca & A. Topa Gomes (U.Porto, Portugal) & R. Coutinho (UFPE, Brazil)
The residual soils present a cohesive-frictional nature with its shear strength significantly influenced by:

1. The presence of bonding between particles, that gives both a component of strength and of stiffness;
2. Widely variable void ratio which is a function of the weathering process and is not related to stress history; and,
3. Partial saturation, possibly to considerable depth.

**BEING THE RESISTANCE RULED BY A PAIR OF FACTORS, WILL IT BE POSSIBLE TO DERIVE IT FROM TESTS GIVING A SOLE PARAMETER?**
The cohesive-frictional nature of residual soils

A. Viana da Fonseca & A. Topa Gomes (U.Porto, Portugal) & R. Coutinho (UFPE, Brazil)

Mayne, 2001
Is it feasible to characterize cohesive-frictional soil with $N_{SPT}$ or $q_c$?

Application of a frictional relation between $q_c$ from CPT and $\sigma'_v0$ in a residual soil profile reveal a moderate increase of $q_c$ in depth.

...higher values of $\phi'$, especially at lower depths, than those obtained from triaxial tests, since the effective cohesive component is not considered.

A wide range of friction angles, a sign of a cohesive strength component.

Robertson and Campanella (1983)
Can flat dilatometer tests (DMT) be used for that purpose?

TWO basic independent parameters ($P_0$ and $P_1$) ->

- possible to derive TWO strength parameters, the angle of shear resistance and the effective cohesive intercept.

A multiple parametrical approach to DMT test has been proposed by Cruz and Viana da Fonseca, by mean of OCR (DMT) or $M/q_c$.

Empirical evaluation of $c'$ from "virtual" OCR (with lab tests, over representative soils)

\[ y = 0.3687x + 3.7728 \quad R^2 = 0.9137 \]

\[ y = 0.9293x + 6.5504 \quad R^2 = 0.7398 \]
Geomechanical Characterization – Deformability

Typical Pressuremeter curves in a Residual Soil – Suction Effect

a) Typical suction monitored pressuremeter tests in a granitic residual soil from Brazil (Schnaid et al., 2004).

b) Typical curve fitting of SMPMT (after Schnaid et al., 2003).
Mechanical Properties: Deformability (saturated residual soils)

HIGH VALUES OF MAXIMUM (ELASTIC) STIFFNESS

\[ G_0 = \rho \cdot V_s^2 \]

\( V_s \), Shear seismic wave velocity

Comparison between observed and reference proposals of \( G_0 \) variation with effective stress (all residual soils)

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Mechanical Properties: Deformability (saturated residual soils)

High $G_0$ derived from in situ tests in routine analysis

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

- High values of constant “C” or “C” (cement implies high stiffness);

- Low values of the exponent “n” or “n” (confinement stress does not prevail towards cement bonding)
Mechanical Properties: Deformability (saturated residual soils)

Degree of non-linearity – Degradation of maximum shear modulus \((G_0)\)

Modified hyperbola by Fahey and Carter (1993), in mobilized strength or stress level \((q/q_u)\):

\[
\frac{E}{E_0} = 1 - f \left( \frac{q}{q_{ult}} \right)^g
\]

- \(f\) controls the strain for the peak strength and \(g\) the shape of the degradation law, being function of the stress level, the inverse of safety factor.

Values of \(f=1\) and \(g=0.3\) for monotonic loading conditions, in uncemented geomaterials...

**MUCH HIGHER VALUES ARE EXPECTED FOR** \(g\)... **IN STRUCTURED SOILS**
Shear modulus versus shear stress level: results of resonant column tests in residual soils from granite of Porto versus data from other testing conditions in sandy soils

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Mechanical Properties: Deformability (saturated residual soils)

LOWER RATE OF NON-LINEARITY IN LATERITIC SOILS

Saprolitic soils versus lateritic (mature) residual soils; these are ruled by secondary cementation processes (Pinto e Abramento, 1997)
A good definition of constitutive laws may rely on

- The availability of high quality samples, generally difficult to get, for triaxial tests with local instrumentation, with high accuracy;

- The simultaneous use of the information of seismic tests for the evaluation of $G_0$ and of the variation of stiffness with stress-strain levels in high quality tests ($G_s$ versus $\gamma$);

  - adjustment of the dynamic or initial shear modulus in laboratory ($G_{0,lab}$) – with RC or BE tests – towards the reference value evaluated in situ ($G_{0,in situ}$), and

  - fundament fitting of the shear modulus deduced from other tests, when available, such as SBPT, trying to integrate them in the constitutive law.
Mechanical Properties: Deformability (saturated residual soils) Overall fitting of SBPT and PMT pressure-expansion curves An optimum approach to characterize residual soil!

Application of cavity expansion theories (Yu & Houlsby, 1991; 1995; Schnaid & Mántaras, 2003) to high quality SBPT were presented by Schnaid et al. in ISC2.

This is made with a curve fitting technique interpreting both the loading and unloading pressuremeter curves, converging to the strength and deformation parameters in residual soil.

In situ test results and analytic simulation in gneissic residual soil (Schnaid, & Mantaras, 2004)
Fitting pressuremeter tests (SBPT or PMT) curves ...
An optimum approach to characterize residual soil!

For drained conditions the angle of shearing resistance can be estimated by Hughes et al. approach: Mohr-Coulomb plastic behaviour and zero cohesion, with a constant dilation angle ($\Psi$):

\[
\sin \phi' = \frac{S}{1 + (S - 1)\sin \phi'_{cv}}
\]

\[
\sin \psi = S + (S - 1)\sin \phi'_{cv}
\]

New cavity expansion model that incorporates the effects of structure and structure degradation into cylindrical cavity expansion theory, was proposed by Mántaras & Schnaid (2002) and Schnaid & Mántaras (2003).
Fitting pressuremeter tests (SBPT or PMT) curves ... An optimum approach to characterize residual soil!

It is really a fitting process with an optimization technique; others...

start from results derived from lab tests and cross-hole seismic tests, applying a “distorted” hyperbolic model, in numerical programs (ex. CAMFE, Fahey & Carter, 1993), to find the parameters that may give the best fit to the observed results...
Specific Features in Residual Soils

- At rest stress state
- Permeability
- Collapsibility
At rest stress state in residual soils
Still an unsolved issue !!!

- $K_0$ from semi-empirical correlations are clearly misleading in residual soils, because of their cemented microstructure.
- The sensitivity of natural soils, especially of residual cemented soils, makes this task (even for SBPT) very unreliable when skilfulness is not present - expressed in non reliable values of lift-off pressure...

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At rest stress state in residual soils
Can we solve it by fundamental ratios?

The use of shear wave velocities determined in Down-Hole ($V_{svh}$) and Cross-Hole ($V_{shv}$) tests for the evaluation of $K_0$

$$
\left( \frac{V_{svh}}{V_{shv}} \cdot \frac{C_{shv}}{C_{shh}} \right)^{\frac{1}{n}} = K_0
$$

$$
K_0 = \left( \frac{V_{svh}}{V_{shv}} \right)^{\frac{1}{n_b-n_a}}
$$

However, an important subject should be taken into consideration and this is the dual and independent factors that rule the dependence of $V_{shh}$, $V_{shv}$, or, $V_{svh}$ on the level of (micro-) structuring and stress state.

$$
V_{svh} = C_{svh}^{vh} + C_{sdh}^{vh} \cdot \sqrt{F(e)} \cdot \sigma_v^{n_a} \cdot \sigma_h^{n_b}
$$

This concept, jeopardizes the direct and simplified use of the relations expressed above!
At rest stress state in residual soils
Can we solve it by correlations?

The usefulness of combining CPT(U) + DMT has been proposed with the following correlation to derive $K_0$ in granular sedimentary soils:

$$K_0 = C_1 + C_2 \cdot K_D + C_3 \cdot \frac{q_c}{\sigma'_v}$$

There have been some proposals to correct $C_2$ constant of the previous equation with interesting results for residual soil, but very much empirical:

![Graph showing $K_0$ vs. Depth for residual soil]
At rest stress state in residual soils
IN NON SATURATED CONDITION !!!

Estimation of $K_0$ by specific correlations for residual soils
(Cunha & Vecchi, 2001)
Collapsibility: identification and classification of collapsible residual soils

(Dourado & Coutinho, 2007)

Classification of the soil collapsibility according Jennings & Knigth (1975) and variation of the $C_{\text{press}}$ versus depth

Pressuremeter tests in a collapsible soil
(see proposal of classification)
Collapsibility: Settlement due to soaking (collapse)

(a) In situ collapse test with plate load tests (PLT); (b) Typical result of the in situ collapse test with "expansocollapsometer" (ECT), Souza Neto et al (2005) and Coutinho et al. (2004)
PARTIAL CONCLUSIONS

Residual soils are a product of rock mass weathering and complex diagenesis that generated materials dominated by strong inhomogeneity, fabric and structure (macro and micro), and generally in unsaturated condition.

Emphasis in this lecture was made in geophysical methods for mapping, zonation and classification.

Structure, specially the effect of interparticle bonding, has a significant influence in the interpretation of test data, especially from in situ tests.

The importance of sample quality, although difficult, was “stressed” as pre-request of good characterization.
Singularities of the behaviour of residual soils were addressed in the lecture giving relevance to the consequences in very objective parameters, useful to geotechnical design, such as:

- at rest stress state;
- stiffness, since the very small strain levels to serviceability levels;
- non-linearity of stiffness or degradation laws;
- cohesive-frictional nature of strength; and,
- special features, such as permeability, collapsibility and ñsat state

The special features that demand for unusual or non-classical approaches in the characterization of residual soils are recognized to be the key point for subsequent analysis and future studies.
Self-Boring Pressuremeter Tests in Porto Residual Soil: Results and Numerical Modelling

- < 10% clay, 20% silt, 70% sand
- Reduced or zero plasticity
- Typical shear strength parameters:
  - $\phi'_c \approx 34^\circ - 36^\circ$; $c' \approx 0 - 50\text{kPa}$; $\phi' \approx 39^\circ - 45^\circ$; $E \approx 30 - 200\text{ MPa}$

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SBP Tests in Porto Residual Soil

- “Self-boring” difficult in these soils
- Requires “tricone” rock roller bit, set forward of the cutting shoe

“Overdrilling” (almost) inevitable

Effect on test quality
SBP Tests – Location and objectives

- **AIM:** retrospective analysis of “Salgueiros” station excavation for the Porto Metro
  - Compare “prediction” with performance
  - Extract the maximum possible amount of design information from the SBP tests by using FE modelling to fit a non-linear elastic Mohr Coulomb plastic model to the test results

- **EXCAVATION SUPPORT:** Novel solution taking full advantage of the soil arching effect
  - Estimation of $K_0$ values vital for the design
  - Required definition of *in situ* of stiffness and strength parameters for design
  - Soil heterogeneity important, both vertical and in plan
SBP Tests – “Salgueiros” Station

- The solution: retaining structure exploiting soil arching effect

Twin rail tunnel
SBP: Test Locations

- Site and tests performed

![Diagram of test locations](image)
### SBP Test Details

- **Tests performed** – 2 locations – 11 tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Depth (m)</th>
<th>$\sigma_{v0}$ (kPa)</th>
<th>$\sigma'_{v0}$ (kPa)</th>
<th>$u_0$ (kPa)</th>
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<tbody>
<tr>
<td>P1-T1</td>
<td>3.85</td>
<td>69.3</td>
<td>69.3</td>
<td></td>
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<tr>
<td>P1-T2</td>
<td>4.90</td>
<td>88.2</td>
<td>88.2</td>
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<td>7.50</td>
<td>135.0</td>
<td>135.0</td>
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<td>298.8</td>
<td>214.4</td>
<td>84.37</td>
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Soil Parameters from the SBP Tests

- Preliminary tentative direct determination of soil parameters using “standard” methods
- Strength parameters estimated by Hughes et al. (1977) procedure for non-cohesive materials (assumes $c' = 0$)

<table>
<thead>
<tr>
<th>Test</th>
<th>$S$</th>
<th>$\phi'$ (°)</th>
<th>$\psi$ (°)</th>
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<tr>
<td>P1-T1</td>
<td>0.63</td>
<td>52.7</td>
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<td>P1-T2</td>
<td>0.51</td>
<td>44.5</td>
<td>13.4</td>
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<td>P1-T3</td>
<td>0.56</td>
<td>48.1</td>
<td>18.5</td>
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<tr>
<td>P1-T4</td>
<td>0.54</td>
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<td>P2-T1</td>
<td>0.44</td>
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<td>0.51</td>
<td>44.4</td>
<td>13.3</td>
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<td>0.51</td>
<td>44.8</td>
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<td>7.1</td>
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<tr>
<td>P2-T6</td>
<td>0.52</td>
<td>45.5</td>
<td>14.8</td>
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</table>

Values tend to be too high! ⇒ Importance of $c'$
Soil Parameters from the SBP Tests

- **Estimation of** $K_o$ : direct determination from the lift-off pressure
- Effect of “over-drilling” significant
- Attempt to obtain $K_o$ using a procedure similar to the one used for Ménard pressuremeter
- **REDUCED CONFIDENCE IN THE VALUES OBTAINED**
Numerical Modelling of the SBP Tests

- Model based on a “distorted” hyperbolic stress-strain relationship (Fahey & Carter, 1993: “f-g model”)

- Model parameters to vary:
  - $c', \phi', \psi, n$ (Mohr-Coulomb + Poisson’s ratio)
  - $C, n$ (give $G_0$ in terms of mean stress, $p'$)
  - $f, g$ (parameters that control the non-linearity of the stress-strain behaviour)

- **Laboratory tests** provide limits to the strength parameters and fix the value of $\phi$ (related to the strain to peak strength)

- **In situ cross-hole** tests define $G_0$ with depth
Numerical Modelling of the SBP Tests

- A good fit was achieved for almost all the tests

Adjust input parameters to obtain good overall fit, including similar $G_{ur}$ values
Good fit using higher $K_0$ than generally used in Porto, at least for less-weathered materials

<table>
<thead>
<tr>
<th>Test</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (°)</th>
<th>$\psi$ (°)</th>
<th>C</th>
<th>n</th>
<th>g</th>
<th>$\mu$</th>
<th>$K_0$</th>
<th>$G_0^*$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1-T1</td>
<td>10</td>
<td>39</td>
<td>5</td>
<td>1050</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.57</td>
<td>91.3</td>
</tr>
<tr>
<td>P1-T2</td>
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<td>39</td>
<td>10</td>
<td>1220</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.74</td>
<td>111.8</td>
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<tr>
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<td>0</td>
<td>1125</td>
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<td>0.3</td>
<td>0.3</td>
<td>0.6</td>
<td>107.6</td>
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<tr>
<td>P1-T4</td>
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<td>43</td>
<td>15</td>
<td>1275</td>
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<td>0.2</td>
<td>0.3</td>
<td>0.73</td>
<td>131.8</td>
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<tr>
<td>P1-T5</td>
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<td>43</td>
<td>10</td>
<td>2000</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.7</td>
<td>206</td>
</tr>
<tr>
<td>P2-T1</td>
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<td>45</td>
<td>15</td>
<td>3200</td>
<td>0.5</td>
<td>0.3</td>
<td>0.3</td>
<td>0.8</td>
<td>270.4</td>
</tr>
<tr>
<td>P2-T2</td>
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<td>45</td>
<td>15</td>
<td>2750</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>1.4</td>
<td>267.8</td>
</tr>
<tr>
<td>P2-T3</td>
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<td>15</td>
<td>4800</td>
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<td>0.3</td>
<td>0.3</td>
<td>1.2</td>
<td>474.3</td>
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<tr>
<td>P2-T4</td>
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<td>45</td>
<td>15</td>
<td>3900</td>
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<td>0.3</td>
<td>1.33</td>
<td>691.2</td>
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</tbody>
</table>
A consistent pattern in the final results was achieved, indicating that the methodology was appropriate to model the SBP tests.

Strength parameters deduced were consistent with laboratory values with high quality samples.

The $G_{ur}$ values obtained from the SBP tests were consistent with the CH tests because of the effects of "over-drilling" in the SBP tests, none of the classical methodologies used to obtain $K_0$ seemed absolutely reliable.

The numerical results suggested higher $K_0$ values for the less weathered materials than used in design in Porto.
Is it really possible to perform “perfect” SBP tests in granular materials with coarse grains as in granite residual soils?
- Is there always some drilling disturbance (overdrilling)?

Nevertheless, even in such cases, results of SBP tests can give useful geotechnical design information if their interpretation is combined with good laboratory tests and numerical modelling.

Next stage is to carry out full 3-D modelling of the excavation using the derived stress-strain parameters.
- Can behaviour in Porto residual soil be “predicted” using parameters derived using the approach described?
THANK YOU FOR YOUR ATTENTION!

MERCI DE VOTRE ATTENTION!