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Versatile site characterization by seismic piezocone

De la souplesse d'emploi du piézocône sismique pour la caractérisation de site

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ABSTRACT
Site investigations require the collection of geostratigraphic information about the soil layering and consistency, as well as a suite of soil properties and parameters for use in analytical and numerical calculations for design. The seismic piezocone test with dissipation phases (SCPTÙ) offers an optimal means to obtain up to five independent readings on soil behavior within a single sounding (q_t, f_s, u_b, t_{50}, V_s). Therefore, this test should be adopted as the minimum level of effort during geotechnical site exploration as site-specific delineation of soil strata is accomplished along with rational means to measure the in-situ small-strain stiffness and estimate strength, stress history, frictional characteristics, and permeability through site specific or global correlations.

RÉSUMÉ
Les investigations de terrain requièrent la collecte d'informations géostratigraphiques quant à la succession des strates, leur consistance, ainsi qu'un ensemble de propriétés et paramètres du sol, afin d'être utilisés dans des modèles numériques et analytiques de conception. Le test effectué par l'intermédiaire du piézocône sismique à dissipation par phases (SCPTÙ) offre l'énorme avantage d'obtenir jusqu'à cinq valeurs indépendantes relatives au comportement du sol grâce à un seul et unique sondage (q_t, f_s, u_b, t_{50}, V_s). Par conséquent, ce test devrait être adopté comme l'effort minimum à faire preuve lors de l'exploration géotechnique d'un site, lorsque la délimitation des couches du sol pour une zone donnée est réalisée de concert avec des moyens rationnels de mesure *in situ* de la raideur aux petites déformations, en même temps que l'estimation de la résistance, de l'historique des contraintes, des caractéristiques de friction, ainsi que celle de la perméabilité par l'intermédiaire de corrélations globales ou spécifiques au site.

1 INTRODUCTION

Site characterization is the initial first step towards the solution to any and all geotechnical projects involving foundations, walls, earth dams, pilings, tunnels, and other facilities. The ground conditions comprised of soil materials are complex with enumerable variations in geologic origins, particle size, mineralogy, stress history, geostratigraphy, and age. The best practices for geotechnical explorations available today include a detailed drilling and sampling program, series of in-situ tests, geophysical surveys, and companion sets of laboratory tests on high-quality samples, including reference triaxial, simple shear, oedometer, permeameter, and bender element testing.

Too often, however, the geotechnical investigation relies on a set of simple soil borings by rotary drilling with small drive samples and recorded N-values from the standard penetration test (SPT). Only limited laboratory testing is possible because disturbance remains a problem in collecting "undisturbed" samples of clays. For sands/silts, undisturbed sampling is impossible especially when saturated, except by in-situ freezing and coring, whose costs are beyond the budgets of all but critical projects. More importantly, the single N-value is insufficient for providing the necessary soil input parameters in a routine analytical evaluation (e.g., axial pile response), yet alone for a full simulation by finite element analysis (e.g., PLAXIS, CRISP. ABAQUS) or finite differences (e.g., FLAC).

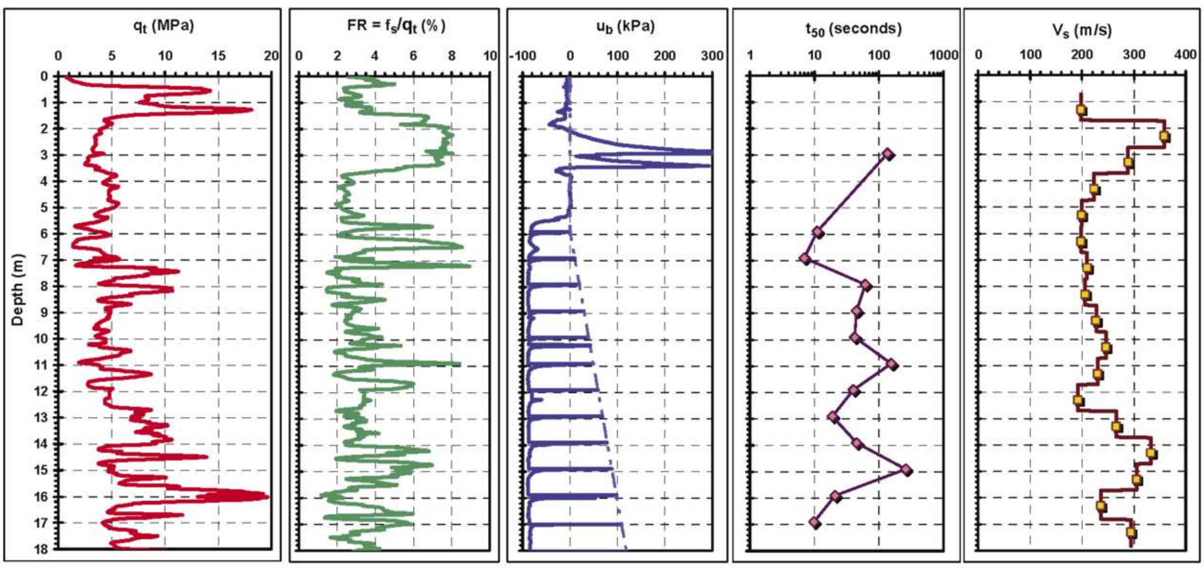

Figure 1. Seismic piezocone sounding with dissipation phases (SCPTÙ) in Piedmont sandy silt residuum at Atlanta airport (Mayne, 2004).

The seismic cone penetrometer was introduced as a hybrid device that could obtain penetration resistance readings together with other geophysical tests (Campanella, et al. 1986). The direct-push technology of cone penetration testing (CPT) facilitates the placement of the accelerometers or geophones to allow measurement of downhole shear wave velocity profiles at one-meter intervals. Excellent coupling between the soil and geophones is achieved. A surface source is used to generate a horizontally-polarized vertically-propagating shear wave that is generated at each rod change. The penetrometer readings are taken at frequent 1- to 2-cm intervals and include: cone tip resistance (q_t), sleeve friction (f_s), porewater pressure (u_1 at the face or u_2 at the shoulder), and inclination from the vertical (*i*). During the temporary halt at 1-m rod intervals, the decay in porewater pressures with time can be measured at selected depths. The time to reach fifty percent consolidation (t_{50}) is a common value for dissipation testing. Taken together, five separate measurements (q_t, f_s, u_2, t_{50}, V_s) of soil behavior can be recorded within the same sounding, termed the seismic piezocone test (SCPTÙ).

A representative SCPTÙ from the new runway 5 expansion at the Atlanta Hartsfield Jackson Airport is shown in Figure 1. Soils consist of fine sandy silt to silty fine sands which are Piedmont residuum derived from the inplace weathering of granitic gneiss and schist. The groundwater table is 5 m deep, as evidenced by both the start of negative porewater pressures (Finke et al. 2001) as well as full dissipations to hydrostatic u_0.

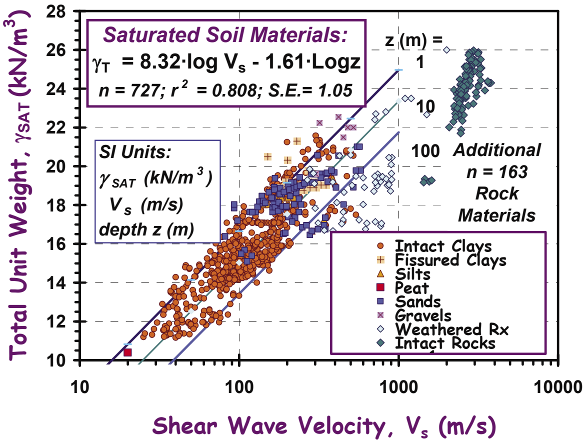

Figure 2. Unit weight relationship with depth z and V_s (Mayne 2001).

2 DELINEATING GEOSTRATIGRAPHY

The continuous readings of the CPT provide fine detailing in the soil profile to define the layering and strata interfaces, as well as presence of clay seams, lenses, and sand stringers. Soil type within each layer can be assessed using approximate "rules of thumb" (e.g., sand if $q_t > 5$ MPa; clay if $q_t < 2$ MPa). For general guidelines, charts interrelating the q_t and friction ratio ($FR = 100 \cdot f_s/q_t$) are available (Robertson & Campanella, 1983), or more detailed soil behavioral classification charts based on all three penetrometer readings (q_t, f_s, u_2) can be used (Robertson, et al. 1986). In fact, all four readings (q_t, f_s, u_2, V_s) can be employed to help better discern "nontextbook" type geomaterials, such as cemented and structured soils. In this case, the shear wave velocity determines the small-strain shear modulus, G_0:

$$G_0 = G_{max} = \rho_T V_s^2 \qquad (1)$$

where $\rho_T = \gamma_T/g_a$ = soil mass density, γ_T = total unit weight, and g_a = 9.8 m/s2 = gravitational acceleration. The ratio G_0/q_t can be graphed versus normalized cone resistance, $Q = (q_t-\sigma_{vo})/\sigma_{vo}'$ to add another level to soil type categorization (Lunne, et al. 1997). In the case of sandy materials, the normalization of cone tip resistance is better represented by $q_{t1} = q_t/(\sigma_{atm}\cdot\sigma_{vo}')^{0.5}$ where σ_{atm} = 1 bar = 100 kPa = atmospheric pressure. In this case, cemented sands or structured residual soils can be identified (Schnaid, et al. 2004).

3 INTERPRETATION OF SOIL PARAMETERS

With as many as five independent measurements on soil response, the SCPTÙ can offer a reasonable assessment on the stress-strain-strength-flow characteristics of each soil layer. However, it must be remembered that the cone penetrometer is an *index* tool. Except for shear wave velocity or elastic modulus which are directly measured, all other soil parameters are estimated from empirical correlations based on theoretical concepts. The best approach is to develop site-specific correlations for clays, e.g., measure representative undrained shear strength by field vane or lab tests and develop correlation parameters for your site or given clay layer. Unfortunately, this is not possible for sandy soils where sampling is not possible and correlations must be based on chamber calibration tests or other in-situ tests. The use of published global correlations is often problematic and should be used with caution, since they vary with geomorphology, mineralogy, drainage, history and undefined measurement errors to name a few, hence, their normal variation is very large.

Selected parameters are discussed below. Detailed interpretations on a full suite of geotechnical parameters from CPT data can be found elsewhere (e.g., Robertson & Campanella, 1983a, b; Jamiolkowski, et al. 1985; Campanella & Robertson 1988; Lunne et al. 1997; Mayne 2004).

3.1 *Small Strain Shear Modulus*

A variety of field and lab tests are available for the determination of the small-strain shear modulus (G_{max}) of soils. Notably, the laboratory tests can be severely affected by sample disturbance and ageing effects. An overview of the field methods is given by Campanella (1994) including techniques as crosshole, downhole, surface waves, refraction, and suspension logging. In the SCPTU, the shear wave profile is obtained in a downhole manner, yet at the convenience of requiring no borehole, casing, or grouting. The use of the penetrometer to deploy the geophones to a subsurface location is efficient and economical in comparison to the conventional geophysical methods. Details on the conduct of the downhole test portion using a pseudo-interval procedure are described by Campanella et al. (1986). The penetrometer can alternatively be fitted with multi-geophone arrangements for conducting true-interval downhole and/or crosshole tests (e.g., Baldi, et al. 1988).

For the calculation of G_{max} from the V_s profile, an assessment of the total unit weight of the soil is needed. Figure 2 shows a relationship for saturated geomaterials that provides an estimate of the unit weight (γ_T in kN/m^3) in terms of depth (z in m) and shear wave velocity (V_s in m/s).

3.2 *Effective Friction Angle*

The effective stress friction angle of soils (ϕ') is a basic strength parameter that is required for geotechnical analyses of bearing capacity, wall pressures, pile side friction, and slope stability. The determination of ϕ' in sands is often related to an inverse deep bearing capacity, as the penetrometer mimics a miniature driven pile. In this case, the well-known relationship for obtaining ϕ' in clean quartz sands that are unaged and uncemented is obtained (Robertson & Campanella, 1983a):

$$\phi' \text{ (deg)} = \arctan [0.1 + 0.38 \log(q_t/\sigma_{vo}')] \qquad (2)$$

Curiosity warrants a look at the use of the CPTU porewater pressures for interpreting ϕ' in fine-grained materials. As such, the NTH method (Senneset et al. 1989) has been developed to utilize the measured cone resistance number ($N_m = \Delta q_{tnet}/\Delta\sigma_{vo}'$) and normalized porewater pressures ($B_q = \Delta u_2/\Delta q_{tnet}$), where $q_{tnet} = q_t - \sigma_{vo}$. For the simplified case where the effective cohesion intercept $c' = 0$, then N_m becomes the normalized cone tip stress, $Q = (q_t\text{-}\sigma_{vo})/\sigma_{vo}'$. Details on the full data processing procedures are given elsewhere (e.g., Sandven, 1990). An approximate approach is shown by lines in Figure 3 for the ranges: $0.1 < B_q < 1.0$ and $20° < \phi' < 45°$ given by the expression:

$$\phi' \approx 29.5° \cdot B_q^{0.121} \cdot [0.256 + 0.336 \cdot B_q + \log Q] \tag{3}$$

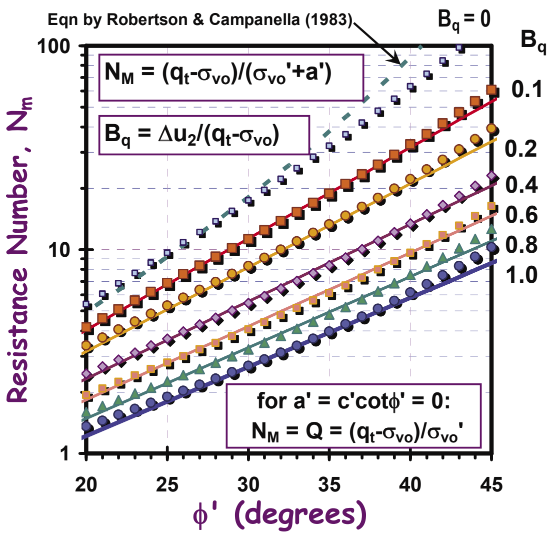

Fig. 3. NTH theory (dots) for effective ϕ' from CPTu results in soils (Senneset et al. 1989); Approximation by lines (Mayne 2004).

3.3 *Preconsolidation stress in clays*

The stress history of clays represents a key facet in determining its behavior under loading. Stress history is demarcated by the preconsolidation stress (σ_p') determined from one-dimensional consolidation tests in the laboratory. The number of consolidation tests for a project is limited in number because of time and budget constraints, as well as issues of sample disturbance. Thus, a direct assessment of σ_p' (backed by lab oedometer testing) can be afforded by piezocone results (e.g., Konrad & Law, 1987; Demers & Leroueil 2002). For a first-order evaluation in intact clays, Figure 4 presents a collection of data compiled from oedometer and piezocone results which indicates:

$$\sigma_p' = 0.33\,(q_t - \sigma_{vo}) \tag{4}$$

The preconsolidation is usefully presented in terms of the normalized overconsolidation ratio, OCR = (σ_p'/σ_{vo}').

3.4 *Undrained shear strength of clays*

For short-term loading of clays and silts, the undrained shear strength (s_u) is often sought for stability and limit state design. In lieu of direct determinations from the CPT data, a critical-state approach can be used (Mayne, 2004):

$$(s_u)_{DSS} = \tfrac{1}{2} \sin\phi' \, OCR^{\Lambda} \, \sigma_{vo}' \tag{5}$$

where the strength mode represents that in direct simple shear (DSS), $\Lambda = 1 - C_s/C_c$ = plastic volumetric strain potential, C_s = swelling index, and C_c = virgin compression index. The value of

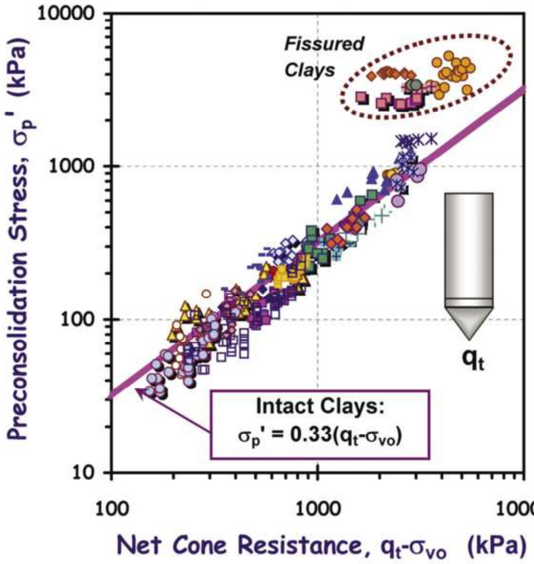

Figure 4. Preconsolidation evaluation from net cone stress in clays (Modified after Kulhawy & Mayne 1990).

Λ is about 0.75 for "normal" clays and increases to about 1 for structured or cemented materials. Modes other than DSS (e.g., triaxial compression, plane strain) can also be accommodated by this approach (Kulhawy & Mayne 1990).

The sensitivity (S_t) of clays may be estimated from the CPTU results (Robertson & Campanella, 1983b). Here, the sleeve friction (f_s) represents the remolded undrained strength, and therefore using (5), the sensitivity becomes:

$$S_t = (s_u)_{DSS}/f_s \tag{6}$$

3.5 *Equivalent Soil Modulus*

Soil response is highly nonlinear from the small-strain region to failure region (Burland, 1989). Therefore, the assessment of an equivalent elastic modulus (E) or shear modulus (G) is complex and requires consideration of the working strain level, confining stresses, and yield envelope. For non-structured, non-cemented, and insensitive geomaterials, a simplified method has been developed for analytical models (Mayne, 2001) to derive stress-strain-strength curves at all depths:

$$\tau = G\,\gamma_s \tag{7a}$$

$$G = G_{max}\,[1 - (\tau/\tau_{max})^g] \tag{7b}$$

where τ = shear stress, γ_s = shear strain, G = secant shear modulus, G_{max} = small-strain shear modulus, τ_{max} = shear strength, and g = empirical fitting parameter for the modified hyperbola. Based on a review of monotonic torsional shear test data (Mayne 2004), the parameter $g \approx 0.3 \pm 0.1$ for "normal" geomaterials, but for structured and/or cemented soils can be 1 or greater. The ratio (τ/τ_{max}) represents the mobilized shear strength, also conveniently considered as 1/FS, where FS = factor of safety. The shear strength is often taken for either undrained conditions ($\tau_{max} = s_{uDSS}$) or for the fully-drained case ($\tau_{max} \approx \sigma_{vo}' \tan\phi'$), although other stress paths are quite possible, as well as complexities associated with partially-saturated soils that are well beyond consideration here.

3.6 *Consolidation Rate and Permeability*

The coefficient of consolidation (c_{vh}) of soils can be evaluated from porewater dissipation tests conducted at select test depths. For fine-grained materials, solutions based in cavity expansion or strain-path methods have been calibrated for use with excess porewater pressures plotted vs. logarithm of time (e.g., Robertson, et al. 1992; Burns & Mayne 1998). Evaluation of a rigidity index ($I_R = G/s_u$) is discussed elsewhere (Mayne 2001). For sandy pervious materials, high-speed monitoring of the porewater decay has been used with Δu plotted vs. square root of time for evaluating c_{vh} (Campanella, et al. 1998).

The coefficient of permeability (k) is linked directly to the coefficient of consolidation via:

$$k = c_{vh} \gamma_w / D' \tag{8}$$

where γ_w = unit weight of water (= 9.8 kN/m^3 for freshwater and 10.0 kN/m^3 for saltwater), and D′ = constrained modulus. A rough estimate of latter can be made from net cone tip resistance (Kulhawy & Mayne, 1990) as: $D' \approx 8(q_t-\sigma_{vo})$. Alternatively, a quick direct means for evaluating hydraulic conductivity k has been proposed by Parez & Fauriel (1988), as given in Fig. 5 and the following trend:

$$k_h\ (cm/s) \approx \left(\frac{1}{251 \cdot t_{50}\ (\text{sec})}\right)^{1.25} \tag{9}$$

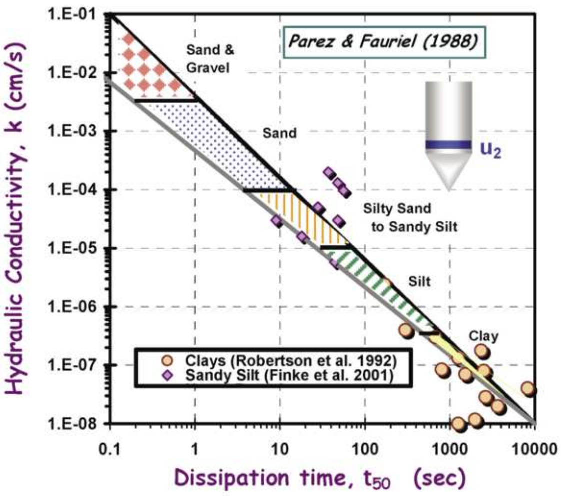

Figure 5. Soil permeability estimate from measured t_{50} dissipation time (after Parez & Fauriel, 1988; Leroueil & Jamiolkowski, 1991).

4 CONCLUSIONS

Site characterization of soils is complex and requires a multitude of both in-situ and laboratory measurements. While suites of field tests can be accomplished using a combination of SPT, CPT, DMT, VST, and PMT, together with various geophysical methods (CHT, DHT, SASW), an expedient and cost-effective method for routine site investigation is the seismic piezocone with dissipation phases (SCPTÙ) as up to five independent readings (q_t, f_s, u_b, t_{50}, V_s) can be obtained at the same location.

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