

ARTICLE DE RECHERCHE / RESEARCH ARTICLE

Developments in the seismic CPT and links to the Ménard Pressuremeter Test

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Abstract – The seismic cone penetration test (SCPT) was developed in the early 1980s and significant developments have been made in its use and application. This paper provides a review of these developments with particular emphasis on its application to identify soil microstructure. The Ménard pre-bored pressuremeter test (PMT) is one of the most popular *in situ* tests in France and links between SCPT results and the PMT are suggested.

Keywords: cone penetration test / seismic test / developments / microstructure / pressuremeter test

Résumé – Développements dans le CPT sismique et liens avec l'essai pressiométrique Ménard. L'essai de pénétration au cône sismique (SCPT) a été développé au début des années 1980 et son utilisation et son application ont fait l'objet de développements significatifs. Cet article fournit une revue de ces développements avec un accent particulier sur son application pour identifier la microstructure du sol. L'essai pressiométrique pré-foré (PMT) Ménard est l'un des essais *in situ* les plus populaires en France et des liens entre les résultats du SCPT et le PMT sont suggérés.

Mots clés : essai au pénétromètre / essai sismique / développements / microstructure / essai au pressiomètre

1 Introduction

The electric cone penetration test (CPT) was developed in the 1960's and has become the most popular *in situ* test worldwide. The basic CPT measurements are the cone tip resistance (q_c) and the sleeve resistance (f_s). Significant developments have occurred over the past 50 years and in the 1970s pore pressure measurements were added. Initially pore pressures were measured on the face of the cone tip (known as the u_1 location) but subsequently the measurement location has moved to the shaft just behind the tip (u_2) and this has become the more popular location. Many cones have the option to measure the pore pressure at either u_1 or u_2 location and some can measure both at the same time. The u_2 location has become popular because it provides the water pressure to make the small net area correction (Campanella and Robertson, 1982) to obtain the corrected total tip resistance (q_t), using the following:

$$q_t = q_c + u_2(1 - a), \quad (1)$$

where a = net area ratio, typically around 0.8.

The CPT with pore pressure measurements is often referred to as a piezocone test or a CPTu. When the CPTu is carried out

in fine-grained soils (e.g., clay and silt), the penetration process is essentially undrained and excess pore pressures (Δu) are generated

$$\Delta u = u_2 + u_o, \quad (2)$$

where u_o = *in situ* equilibrium pore pressure.

The excess pore pressure can be either positive or negative depending on the *in situ* state of the soil. During a pause in the CPT penetration (i.e., a dissipation test) the excess pore pressures will dissipate toward u_o . The rate of dissipation is often recorded by determining the time to reach 50% dissipation (t_{50}), i.e., when 50% of Δu has dissipated. Most CPT probes also measure the inclination (i) of the probe during the test to ensure that the test is performed essentially vertically.

In the early 1980s seismometers (e.g., geophones) were added to include the measurement of the seismic shear wave velocity (V_s) using the downhole method (Robertson *et al.*, 1986; Butcher *et al.*, 2005) and, in recent years, the compression wave velocity (V_p) has also been measured. The seismic CPT is often referred to as a SCPT or SCPTu. The SCPTu is becoming a popular *in situ* test because it can provide 7 or more independent measurements in one cost-effective *in situ* test, such as the following:

- corrected tip resistance, q_t ;
- sleeve resistance, f_s ;

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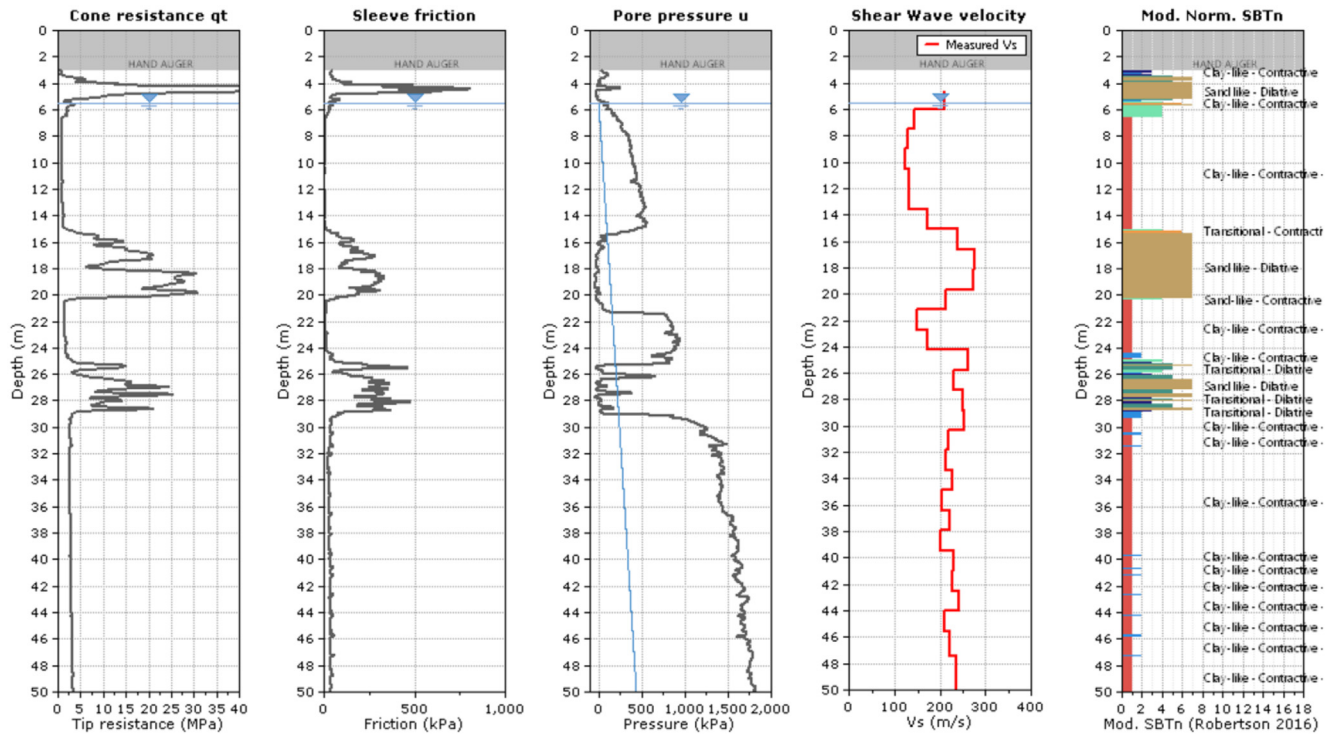


Fig. 1. Example SCPTu profile (San Francisco).

Fig. 1. Exemple de profil SCPTu (San Francisco).

- penetration pore pressures, u_2 and/or u_1 ;
- seismic wave velocities, V_s and V_p ;
- rate of pore pressure dissipation, t_{50} ;
- equilibrium pore pressure, u_0 ;
- probe Inclination, i .

The test provides near continuous measurements of the main parameters (q_t , f_s , u_2) at depth increments from 10 to 50 mm. The other measurements (e.g., t_{50} , u_0 and V_s) are typically made at larger depth increments of 0.5 to 1.0 m or more. The cone is pushed at a rate of 20 mm/s such that a basic CPTu (q_t , f_s , u_2) to a depth of 30 m can be completed in less than 1 hour. The addition of seismic and dissipation testing slows the overall time depending on the depth interval for the additional tests. A typical SCPTu to a depth of 30 m with V_s at 1 m increments plus one dissipation test can typically be completed in less than 2 hours. In developed countries where labor costs are high, the cost of an *in situ* test is closely linked to the total time required to perform the test. Hence, a rapid test, such as the CPTu and SCPTu can be very cost effective. A SCPTu to a depth of 30 m, with data collected every 10 mm, produces a profile with 9,000 data points (i.e., 3,000 each of q_t , f_s and u_2), plus an additional 50 measurements of V_s . With engineering design embracing uncertainty and applying more probabilistic analyses (e.g., Lacasse *et al.*, 2019), the large amount of data provided by the SCPTu supports greater application of these methods.

Figure 1 shows an example of a 50 m SCPTu profile in San Francisco. The V_s profile is shown as a step function to represent the value over the measured depth increment.

A common question related to the CPT and SCPT is “how deep can you push?”. The answer depends on the strength of the ground and the total reaction force available to push the cone (probe). It is common for commercial CPT pushing equipment to provide a maximum push force of about 200 kN at the ground surface. The force available on the cone tip will be a function of the friction developed along the probe and push rods. Typically push rod friction is reduced by using an enlarged section a short distance behind the cone (i.e., friction reducer) to push the soil away from the push rods and hence reduce the total rod friction. Most cones are 10 cm² in cross-sectional area with an approximate 15 cm² friction reducer pushed with standard 10 cm² push rods. In North America, it has become common to use a 15 cm² cone with the standard 10 cm² push rods with no friction reducer, since the cone is already larger than the push rods. This combination can allow the CPT/SCPT to be pushed to greater depth for a given surface reaction. A 15 cm² cone, along with the traditional 10 cm² cone, is accepted in most national and the international standards for the CPT. Lubrication (e.g., drilling mud) has also been used to reduce rod friction for deep soundings. The shorter the probe, the smaller the total rod friction. This has been a major reason for the popularity of the SCPT using a single seismometer (details in a later section) since the probe can maintain a short length (< 500 mm). If rod friction can be minimized, it is possible to push the CPT/SCPT into relatively hard soils (i.e., very dense sands and very stiff clays with undrained strengths up to 1 MPa). In soft ground it is common to push the CPT to a depth of about 100 m. Recent developments have also allowed CPT to greater depths using

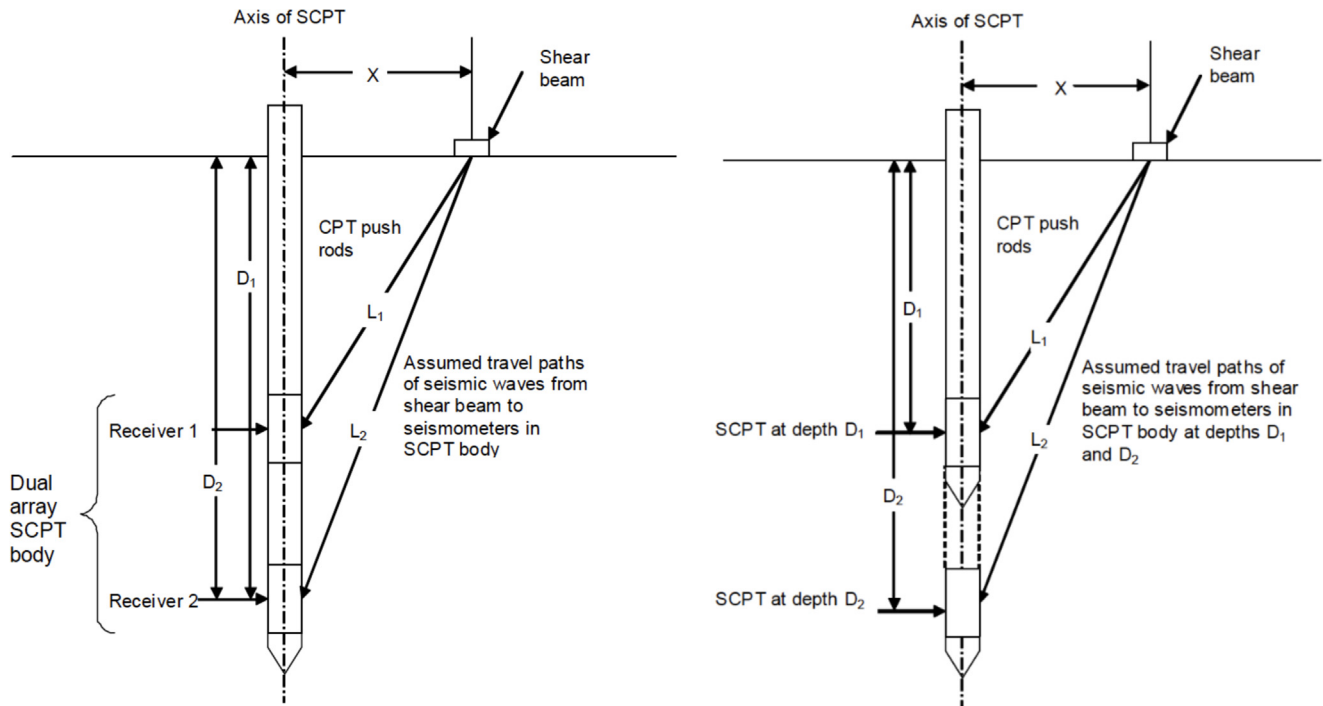


Fig. 2. Schematic of (a) true-interval method, (b) pseudo-interval method (modified from Butcher *et al.*, 2005).

Fig. 2. Schéma de la méthode (a) du vrai intervalle, (b) du pseudo-intervalle (modifié d'après Butcher *et al.*, 2005).

down-hole wireline techniques. Pushing the CPT into gravel deposits can be difficult depending on the density and grain size of the gravel. If the mean grain size is less than about 10 mm it should be possible to push the CPT. When the mean grain size is close to diameter of the probe (*i.e.*, 36 mm for 10 cm² and 44 mm for 15 cm²), refusal is common. Methods have been used to improve the ability to push the CPT into very dense soils and/or gravels (*e.g.*, Sanglerat *et al.*, 1995).

The objective of this paper is to provide a review of the main developments in the use and application of the SCPTu with particular emphasis on its application to identify soil microstructure and to provide links to the parameters measured using the Ménard pre-bored pressuremeter test (PMT).

2 Developments in SCPT methodology

Prior to 1981, shear (V_s) and compression (V_p) wave velocities were often measured using traditional down-hole and cross-hole borehole geophysical methods. These methods require either 1 borehole (for down-hole) or 3 boreholes (for cross-hole) and are expensive and time-consuming. Around 1980, Prof. K. Stokoe suggested that the down-hole method could be done using a seismometer that is push-into the ground rather than using a borehole. In 1981, Prof. R. Campanella added geophones to a piezo-cone penetrometer so that the downhole measurement of shear wave velocity (V_s) could be included with the CPTu. This was the first seismic SCPTu (Rice, 1984, Robertson *et al.*, 1986). This first SCPTu had two geophones 1 m apart behind a CPTu probe to measure V_s using a true-interval approach, as illustrated schematically in Figure 2a. During the initial testing it was observed that the same results could be obtained using only one geophone and a

pseudo-interval approach with the test repeated at 1 m intervals, as shown schematically in Figure 2b. The simple pseudo-time interval approach became popular since it only required a single geophone to be installed in the cone resulting in a simple shorter penetrometer. However, in recent years the true-interval approach has proven to provide more reliable results, has a simpler test procedure and can provide direct determination of V_s in the field without the need for post-processing of the seismic data. A summary of the main advantages and limitations of both true-interval and pseudo-interval methods are summarized as follows.

2.1 True-interval method

Advantages:

- only requires a single shear wave source;
- independent of seismic source trigger characteristics;
- obtains V_s directly in the field.

Limitations:

- probe is longer and harder to push to required depths;
- seismometers must have identical response characteristics (natural frequencies, calibration, and damping);
- if signals are stacked, trigger must be repeatable.

2.2 Pseudo-interval method

Advantages:

- probe can be shorter and easier to push to required depths.

Limitations:

- requires two seismic waves for depth D_1 and D_2 ;

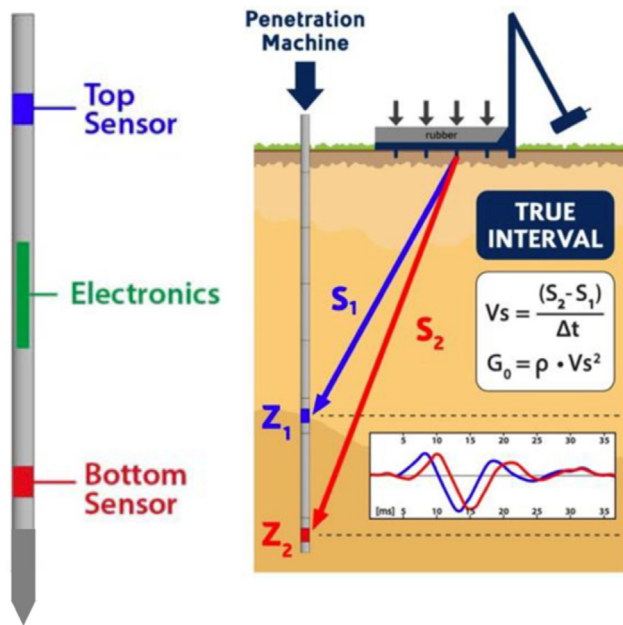


Fig. 3. Example SCPTu true-interval module (modified from Marchetti, 2022).

Fig. 3. Exemple du module SCPTu du vrai intervalle (modifié d'après Marchetti, 2022).

- requires fast and repeatable trigger;
- challenging to determine V_s in the field and requires post-processing.

An example of a modern true-interval SCPTu probe is shown in Figure 3 and Figure 4 illustrates an example data acquisition for a true-interval SCPTu that shows that V_s is measured directly in the field using cross-correlations and can determine measurement repeatability from multiple repetitions from the seismic source.

The seismic source can be a simple horizontal beam that is struck in a horizontal direction to produce a shear wave and flat plate struck in the vertical direction to produce a compression wave. Mayne and McGillivray (2008) created an automatic source that strikes an anvil using a spring-loaded rotating hammer. The automatic hammer is compact, simple, and repeatable. Commercial automatic hydraulic hammers have also been in use for at least 10 years and often use an internal hydraulic cylinder to strike two plates in both directions and generate horizontal reverse polarity waves for cones that have a single seismometer and use the pseudo-interval method. Near continuous seismic measurements have also been made using the SCPTu (e.g., Ku et al., 2013). A summary of the various SCPT methods is illustrated in Figure 5.

Developments have also been made using different seismometers, such as accelerometers. Accelerometers are smaller and sensitive to wide range of frequencies but often require time-consuming post-processing. Seismometer packages have also increased to include either bi-axial (X, Y direction) or tri-axial seismometers packages (X, Y, Z). The bi-axial packages avoid the need to orientate the probe with the seismic source to improved signal quality. The tri-axial packages aid in the addition of V_p measurements.

Data acquisition systems have also evolved to include faster computers with larger memory, automatic cross correlation (true-interval method) and improved reliability.

Several publications have discussed the accuracy of V_s measurements using the SCPT (e.g., Stolte and Cox, 2020). The main conclusions are that if the offset distance between the probe and the seismic source (X, see Figs. 2 and 5) is small (typically less than 1 m), any errors due to non-linear travel path are very small for V_s . The offset distance is often larger for V_p measurements when using horizontally orientated seismometers. Experience (e.g., Styler et al., 2016) has also shown that having a repeatable seismic source can significantly improve data quality and accuracy. In general, the pseudo-interval method is usually within 10% of the true-interval method and has remained a popular and cost-effective method for the SCPT, since the probe is shorter and can penetrate to a greater depth for a given reaction push-force. However, the true-interval method is gaining in popularity due to improved accuracy and real-time determination of seismic velocity and repeatability (see Fig. 4). The trend is to use the true-interval method with sensors 0.5 m apart for a more detailed seismic profile and to keep the length of the probe relatively short. The true-interval method also has the advantage that velocity can be calculated in the field without the need for time-consuming post processing.

The addition of measuring the compression velocity (V_p) has also evolved and improved but the measurement of V_p is generally less reliable compared to V_s . V_p is often used to infer *in situ* saturation where accuracy is less important since V_p in soft soils with 100% saturation is around 1500 m/s and drops rapidly to around 500 m/s at a saturation level of 99%. Hence, accuracy and repeatability are only needed to distinguish between the fast velocity in saturated soils and the much lower velocity in near-saturated or un-saturated soils. The application of V_p to determine *in situ* saturation is not always helpful, since loose contractive soils that have an *in situ* saturation of around 80 to 90% (i.e., small, occluded bubbles) can still behave in an essentially undrained manner when sheared monotonically.

There have been numerous publications showing that V_s profiles measured using the SCPT are in very good agreement with traditional, but more costly, down-hole, cross-hole and surface methods. The newer surface method of Multi-channel Analyses of Surface Waves (MASW) is becoming a popular method to create 2D profiles of both V_s and V_p . However, the interpretation of MASW is not always unique and can be improved significantly with several SCPTu profiles to ground truth the results. It is becoming common to perform MASW along several lines at a site followed by selective SCPTu at locations guided by the initial MASW results. For example, if MASW identifies softer soils at one or more locations across a site, SCPTu can be carried out at those locations to provided more detailed profiles with depth.

3 Applications

One of the main reasons for the growing popularity of the SCPTu is the link between V_s and the small strain shear modulus, G_0 , which is the fundamental soil stiffness at small strains and can be determined using:

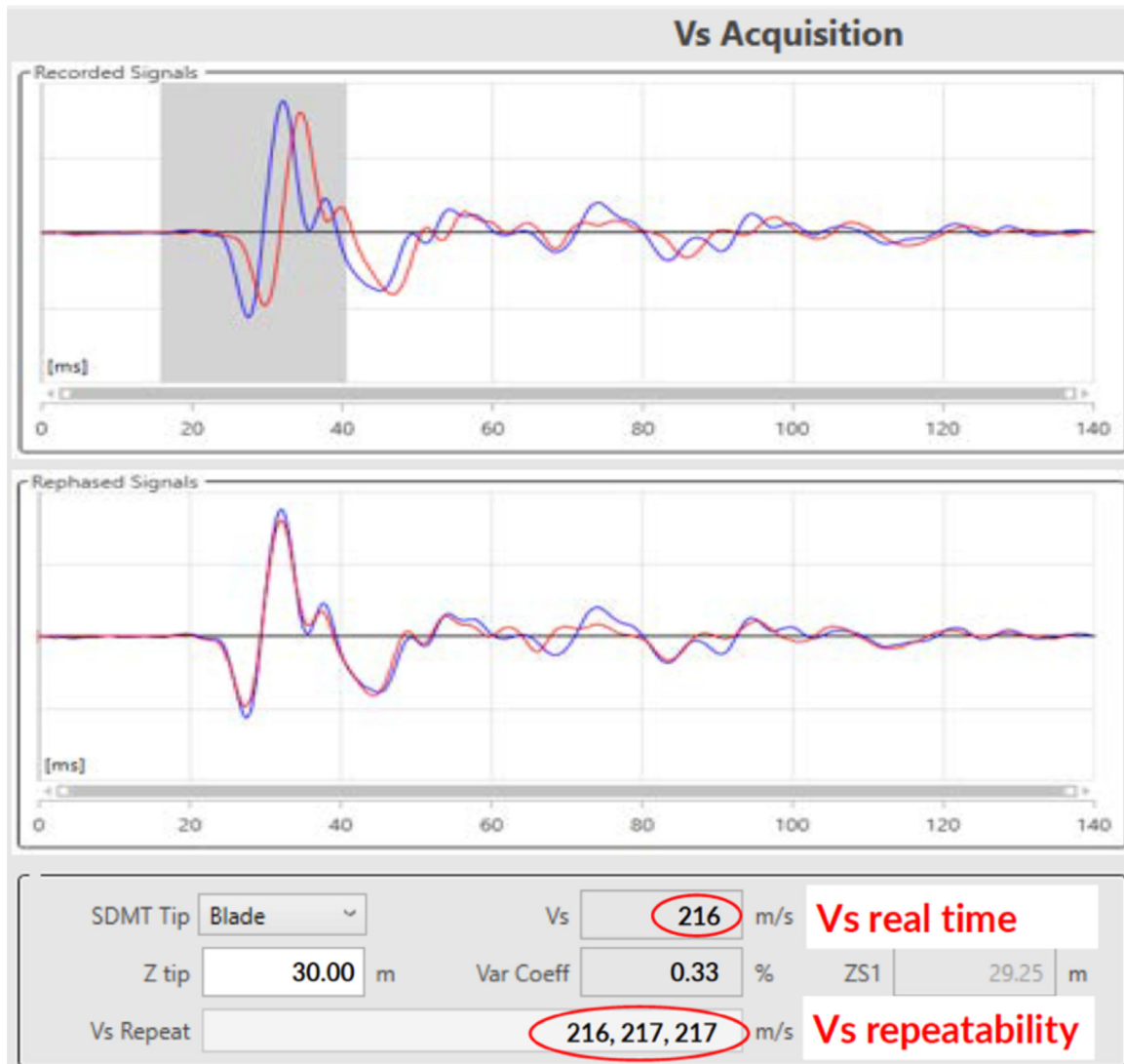


Fig. 4. Example true-time interval SCPTu data acquisition showing two arrival waves from a single source in the upper plot and the cross-correlation shift in the lower plot (modified from Marchetti, 2022).

Fig. 4. Exemple d'acquisition de données SCPTu à intervalle de temps réel montrant deux arrivées d'ondes depuis une seule source (graphique supérieur) et le décalage de la corrélation croisée (graphique inférieur) (modifié d'après Marchetti, 2022).

$$G_o = G_{\max} = \rho_t (V_s)^2, \quad (3)$$

where:

- ρ_T = total soil mass density = γ/g ;
- γ = total unit weight;
- g = gravity.

Hence, the *in situ* measurement of V_s provides a direct measurement of the *in situ* fundamental small strain shear modulus, G_o , which is also independent of drainage conditions.

A summary of the main geotechnical parameters that can be obtained from SCPTu measurements is shown in Table 1.

Example of applications of the SCPTu are summarized in Table 2.

Complete details on CPTu and SCPTu interpretation and applications can be found in a CPT Guide by Robertson and Cabal (2022) as well as Lunne *et al.* (1997).

The following section will focus on the application of SCPTu data to identify soil behavior type (classification) and microstructure.

4 Soil classification and microstructure

One of the major applications of CPT data in general is for soil classification and stratigraphic profiling. The most common CPT-based classification systems are based on behavior characteristics and are often referred to as a Soil Behavior Type (SBT) classification (*e.g.*, Robertson, 1990). Traditional soil classification systems used by most engineers and geologists are based on physical characteristics of grain-size distribution and plasticity (*e.g.*, Atterberg Limits) measured on disturbed and remolded samples. The actual *in situ* soil behavior depends on many other factors such as geologic processes related to origin, environmental factors

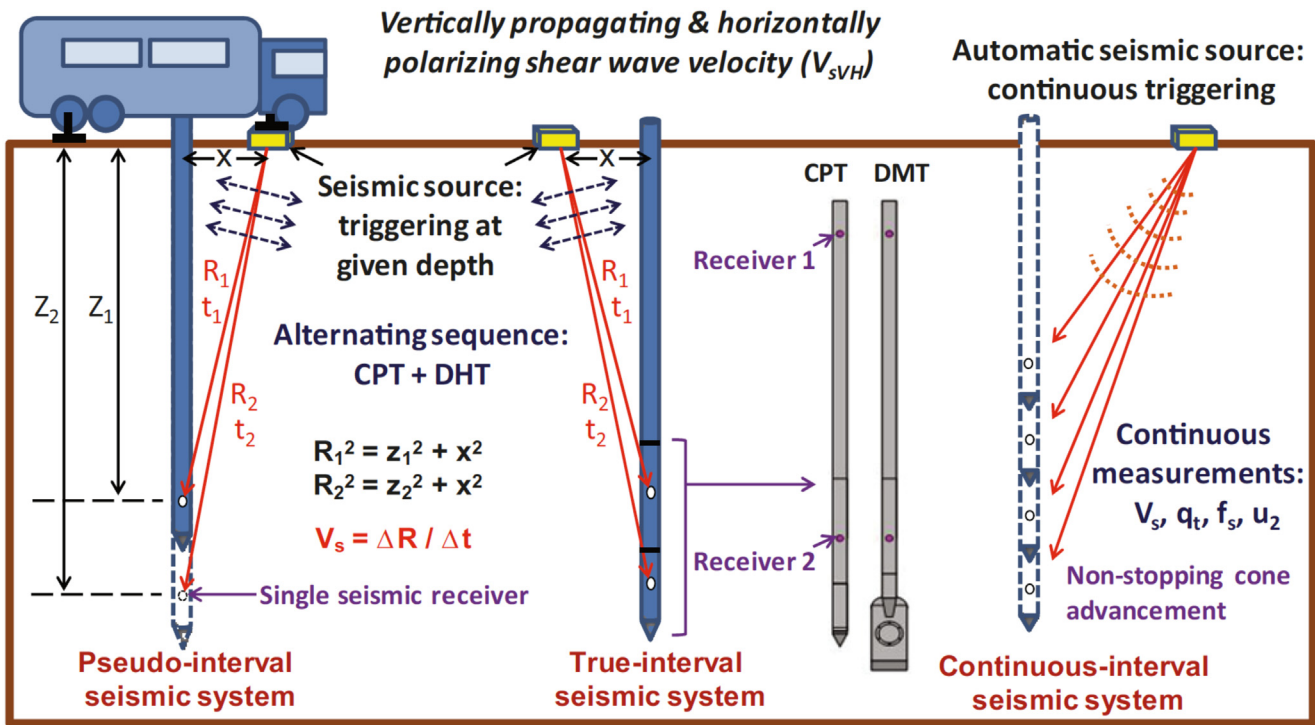


Fig. 5. Shear wave velocity measurement systems using direct-push (CPTu) technology: pseudo-interval seismic system, true-interval seismic system, and continuous-interval seismic system (after Ku *et al.*, 2013).

Fig. 5. Systèmes de mesure de la vitesse des ondes de cisaillement utilisant la technologie de poussée directe (CPTu) : système sismique à pseudo-intervalle, système sismique à intervalle réel et système sismique à intervalle continu (d'après Ku *et al.*, 2013).

Table 1. Summary of methods to estimate geotechnical parameters from SCPTu data.

Table 1. Résumé des méthodes d'estimation des paramètres géotechniques à partir de données SCPTu.

Geotechnical parameter	SCPTu measurements	Example of references
Soil behavior type (SBT)	q_t, f_s, u_2, V_s	Robertson (1990), Robertson (2016)
Unit weight (γ)	q_t, f_s, V_s	Robertson and Cabal (2010), Mayne and Peuchen (2012)
In situ state (ψ , OCR)	q_t, f_s, V_s	Been and Jefferies, (1985), Mayne (2005)
Peak shear strength (ϕ' , s_u)	q_t, f_s, u_2	Lunne <i>et al.</i> (1997), Robertson (2009)
Remolded strength (s_{ur})	f_s	Lunne <i>et al.</i> (1997), Robertson (2009)
Small strain stiffness (G , E)	V_s	Rice, (1984), Robertson <i>et al.</i> , (1986)
Saturation (S)	V_p	Jamiolkowski (2012)
Hydraulic conductivity (k)	t_{50}, V_s	Burns and Mayne (1996)
Coefficient of consolidation (c)	t_{50}, V_s	Teh and Houlsby (1991), Lunne <i>et al.</i> (1997)

(such as stress history) as well as physical and chemical processes. In general, soils tend to become stiffer and stronger with age. The successful link between simple physical characteristics and *in situ* behavior is strongly influenced by geologic factors.

Many natural soils have some form of structure that can make their *in situ* behavior different from those of 'ideal soils'. The term structure can be used to describe features either at the deposit scale (macrostructure), *e.g.*, layering and fissures, or at the particle scale (microstructure), *e.g.*, bonding/cementation. Older natural soils tend to have some microstructure caused by post depositional factors, of which the primary ones tend to be age and bonding (cementation). Many authors have discussed the effects of microstructure

(*e.g.*, Burland, 1990; Leroueil, 1992; Leroueil and Hight, 2003). Microstructure can be caused by many factors such as: secondary compression, thixotropy, cementation, cold welding, and aging (Leroueil and Hight, 2003). Microstructure tends to give a soil a strength and stiffness that cannot be accounted for by void ratio and stress history alone. Leroueil (1992) illustrated the main differences in mechanical behavior between soils with microstructure (*i.e.*, 'structured soils') and 'ideal soils' (*i.e.*, unstructured soils) and showed that for the same 'ideal soil' at the same void ratio, the 'structured soil' with microstructure has higher yield stress, peak strength, and small strain stiffness. At larger strains, when the effects of microstructure can be destroyed due to factors such as compression, shearing, swelling, weathering

Table 2. Summary of applications of SCPTu data.**Table 2.** Résumé des applications des données SCPTu.

Geotechnical application	Example of references
Shallow foundations	Mayne (2000)
Deep foundation	Mayne and Woeller (2008)
Seismic ground response	Hashash <i>et al.</i> (2015)
Resistance to seismic loading (liquefaction)	Andrus and Stokoe (2000), Kayen <i>et al.</i> (2013)
Ground improvement	Massarsch (1991)
Statistical analyses	Lunne <i>et al.</i> (1997)

and fatigue, the soil becomes ‘*destructured*’ (Leroueil and Hight, 2003). The term ‘*ideal soil*’ will be used to describe soils with little or no microstructure that are predominately young and uncemented. The term ‘*structured soil*’ will be used to describe soils with extensive microstructure, such as caused by aging and cementation. Robertson (2016) updated the CPT-based soil behavior type classification system to incorporate descriptions that were more behavior driven and included seismic V_s with CPT measurements to aid in identification of microstructure. The following is a summary of the main points in the Robertson (2016) update.

Critical State Soil Mechanics is based on the observation that soils ultimately reach critical state (CS) at large strains and at critical state there is a unique relationship between shear stress, normal effective stress, and void ratio to define a Critical State Line (CSL). Since critical state is independent of the initial state, the parameters that define critical state depend only on the nature of the grains of the soils and can be linked to basic soil classification (*e.g.*, Atkinson, 2007). The current *in situ* state of a soil can be defined in several ways. In fine-grained soils it is common to define the current state in terms of overconsolidation ratio (OCR) or yield stress ratio, (YSR) that is related to the normal compression line (NCL), since fine-grained ‘*ideal soils*’ tend to have a unique NCL that is essentially parallel to the CSL. In coarse-grained soils it is still common to define *in situ* state in terms of relative density (or density index), especially for clean sands. However, it is becoming more common to define the current state in terms of a state parameter (ψ) that is related to the CSL, since the NCL is not unique (*e.g.*, Been and Jefferies, 1985). At low confining stress the CSL for many clean silica-based sands can be very flat in terms of void ratio *versus* log mean effective stress and hence, there is an approximate link between relative density and state parameter. However, state parameter can capture the current state for most coarse-grained soils over a wide range of stress. There is an approximate link between YSR and state parameter, where $YSR \sim 4$ is similar to $\psi \sim 0$.

There is an important difference between the behavior of ‘*ideal soils*’ that are either ‘*loose*’ or ‘*dense*’ of CS. Soils that are ‘*loose*’ of CS tend to contract on drained loading (or where pore pressures rise on undrained shear). Soils that are ‘*dense*’ of CS tend to dilate at large shear strains (or where pore pressures can decrease in undrained loading). The tendency of soils to change volume while shearing is called dilatancy and is a fundamental aspect of soil behavior.

The behavior of soils in shear prior to failure can be classified into two groups; soils that dilate at large strains and soils that contract at large strains. Saturated soils that contract

at large strains have a shear strength in undrained loading that is lower than the strength in drained loading, whereas saturated soils that dilate at large strains tend to have a shear strength in undrained loading that is either equal to or larger than in drained loading. When saturated soils contract at large strains they may also show a strain softening response in undrained shearing, although not all soils that contract show a strain softening response in undrained shear. This strength loss in undrained shear can result in instability given an appropriate geometry, such as flow liquefaction (*e.g.*, Robertson, 2010). Hence, classification of soils that are either contractive or dilative at large strains can be an important behavior characteristic for many geotechnical problems.

Idriss and Boulanger (2008) classified soils as either sand-like or clay-like in their behavior, where sand-like soils are susceptible to cyclic liquefaction and clay-like soils are not susceptible to cyclic liquefaction. Idriss and Boulanger (2008) suggested that fine-grained soils transition from behavior that is more fundamentally like sand to a behavior that is more fundamentally like clay over a narrow range of plasticity index (PI). Sand-like soils tend to have $PI < 10\%$ and clay-like soils tend to have $PI > 18\%$ (Bray and Sancio, 2006). The transition from more sand-like to more clay-like behavior is conceptually like the transition from coarse-grained (non-plastic) to fine-grained (plastic) soils, although some low or non-plastic fine-grained soils can behave more sand-like. Likewise, the transition from saturated soils that are more sand-like to more clay-like typically corresponds to a response that transitions from one that is predominately drained under most static loading to a response that is predominately undrained under most static loading, although the rate of loading also has a major role.

Any behavior-based classification systems will tend to apply primarily to ‘*ideal soils*’ that have little or no microstructure. Hence, it can be important to have a system and associated *in situ* test that can also provide a method to identify if the soil to be classified has a behavior like most ‘*ideal soils*’, *i.e.*, has little or no microstructure. Since the CPT q_t is a large strain measurement that is a measure of the strength of a soil and V_s (G_o) is a small strain measure of the stiffness of the soils the combined measurements in a SCPT have the potential to identify soils with significant microstructure.

The CPT-based normalized Soil Behavior Type (SBTn) method suggested by Robertson (1990) and updated by Robertson (2016) and Schneider *et al.* (2012) is based on the following normalized CPT parameters:

$$Q_{tn} = [(q_t - \sigma_v)/p_a](p_a/\sigma'_{vo})^n, \quad (4)$$

$$F_r = [(f_s/(q_t - \sigma_{vo}))]100\%, \quad (5)$$

$$B_q = \Delta u/(q_t - \sigma_{vo}), \quad (6)$$

$$U_2 = \Delta u/\sigma'_{vo}, \quad (7)$$

where:

- q_t = cone resistance corrected for water effects;
- σ_{vo} = current *in situ* total vertical stress;
- σ'_{vo} = current *in situ* effective vertical stress;
- q_n = net cone resistance = $q_t - \sigma_{vo}$;
- n = stress exponent that varies with SBTn, and defined by:

$$n = 0.381(I_c) + 0.05(\sigma'_{vo}/p_a) - 0.15, \quad (8)$$

where $n \leq 1.0$ and:

$$I_c = \left[(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2 \right]^{0.5}. \quad (9)$$

As discussed above, a major factor in any classification system can be the effects of post deposition processes that can generate microstructure. Hence, it can be important to first identify if soils have significant microstructure, since this can influence their *in situ* behavior and ultimately the effectiveness of any classification system based on *in situ* tests. Some understanding of the geologic background of the soil is always a required starting point for reliable classification based on CPT data, since geology provides a framework for interpretation.

The SCPTu results include G_o (from V_s) and q_t that represent two different soil measures at opposite ends of the shear-strain curve that share similar dependencies on soil density, stress, soil type, particle properties, and the particle size distribution. The combined information from the SCPT has the potential to aid in identification of possible microstructure in soils to either supplement existing geologic information or when geologic information maybe either lacking or uncertain. [Eslaamizaad and Robertson \(1996\)](#) and [Schnaid \(2009\)](#) suggested that the SCPT can be helpful to identify soils with microstructure based on a link between G_o/q_t and Q_{tn} , since both aging and bonding tend to increase the small strain stiffness (G_o) significantly more than they increase the large strain strength of a soil (reflected in both q_t and Q_{tn}). Hence, for a given soil, age and bonding both tend to increase G_o more than the larger strain cone resistance (q_t), all other factors (*in situ* stress state, etc.) being constant. [Schneider and Moss \(2011\)](#) suggested using an empirical parameter, K_G defined as follows:

$$K_G = (G_o/q_t)(Q_{tn})^{0.75}, \quad (10)$$

where G_o is in same units as q_t and Q_{tn} is dimensionless:

The ratio G_o/q_t is essentially a small strain rigidity index (I_G), since it defines stiffness to strength ratio, where G_o is the small strain stiffness and q_t is a measure of soil strength. [Robertson \(2015\)](#) had suggested that K_G is essentially a normalized rigidity index, since it normalizes the small strain rigidity index (G_o/q_t) with *in situ* soil state reflected by Q_{tn} .

[Robertson \(2016\)](#) suggested that the small strain rigidity index (I_G) can be extended to include fine-grained soils and should be defined based on net cone resistance q_n , since q_n is a more correct measure of soil strength, to be:

$$I_G = G_o/q_n. \quad (11)$$

Hence, a modified normalized small strain rigidity index, K_G^* is defined as:

$$K_G^* = (G_o/q_n)(Q_{tn})^{0.75}. \quad (12)$$

[Figure 6](#) presents a plot of Q_{tn} versus I_G like that shown by [Schneider and Moss \(2011\)](#) but extended to cover a wider range of soils. [Robertson \(2016\)](#) showed that most young, uncemented (*ideal*) silica-based sands have $100 < K_G^* < 330$, with an average value of around 200.

Most of the existing empirical correlations developed for interpretation of CPT results are predominately based on experience in silica-based soils with little or no microstructure (e.g., [Robertson, 2009](#); [Mayne, 2014](#)). Hence, if soils have $K_G^* < 330$ the soils are likely relatively young and uncemented (i.e., have little or no microstructure) and can be classified as ‘*ideal soils*’ (unstructured) where traditional CPT-based empirical correlations likely apply. Soils with $K_G^* > 330$, tend to have significant microstructure and the higher the value of K_G^* , the more microstructure is likely present. Hence, if a soil has $K_G^* > 330$ the soils can be classified as ‘*structured soils*’ where traditional generalized CPT-based empirical correlations may have less reliability and where local modification may be needed. The influence of increasing microstructure on *in situ* soil behavior is often gradual and any separating criteria can be somewhat arbitrary. Data suggests that very young uncemented soils (e.g., hydraulic fills) tend to have K_G^* values closer to 100, whereas soils with some microstructure (e.g., older Pleistocene deposits) tend to have K_G^* values closer to 330. Soils with $K_G^* < 330$ tend to have little or no microstructure where existing empirical CPT-based correlations tend to provide good estimates of soil behavior.

A challenge when calculating K_G^* , is that the CPT parameters (q_t and Q_{tn}) and V_s are often measured over different depth intervals. For example, CPT measurements are typically made at 10 to 50 mm depth intervals, whereas V_s (and hence, G_o) is typically measured over 500 to 1000 mm (or larger) depth intervals. There can be a scale effect when combining the two parameters (G_o/q_n), where the CPT parameters respond to smaller features and variability in the ground and V_s (and G_o) tend to respond in a more subdued average manner. Hence, the calculated value of K_G^* can show some variability in non-homogenous soils.

Natural soils can also be anisotropic where the small strain stiffness can vary depending on the direction of loading where G_{ovH} (and V_{svH}) may not be equal to G_{oHH} (and V_{sHH}). The subscript VH applies to stiffness that is measured in the vertical and horizontal direction, which is the most commonly measured *in situ* shear wave velocity (V_{svH}), i.e., either a vertically propagating wave with particle motion in the horizontal direction (V_{svH}) or a horizontal propagating wave with particle motion in the vertical direction ($V_{shV} = V_{svH}$). The suggested relationship shown in [Figure 6](#) is based on G_{ovH} (and V_{svH}) that is measured primarily using the SCPT. For

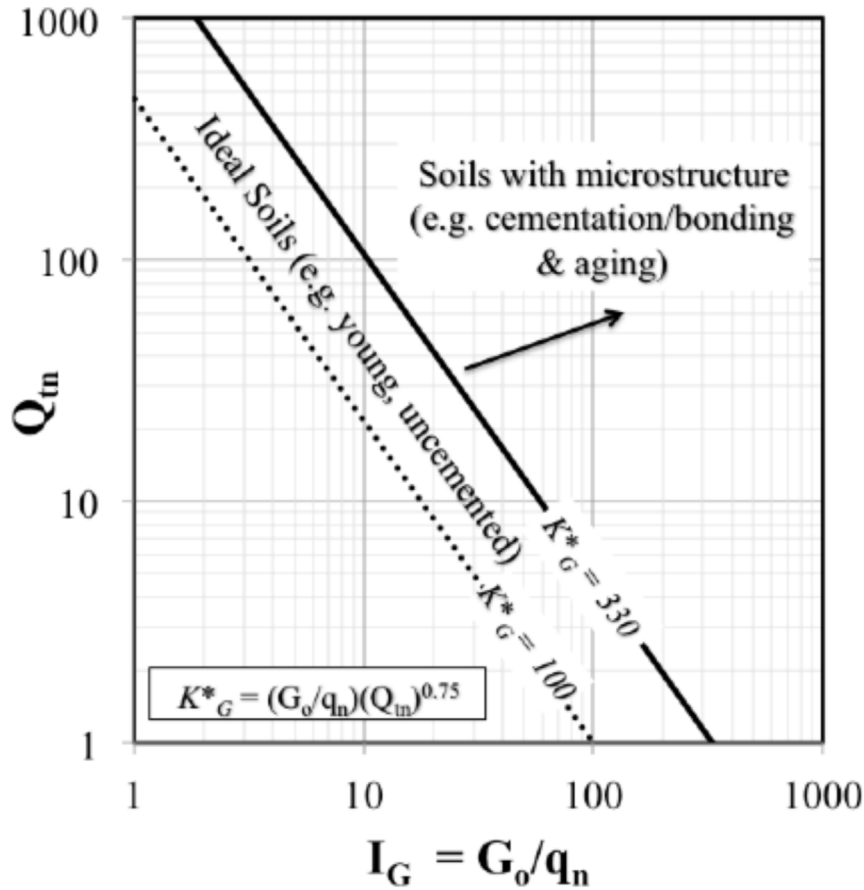


Fig. 6. $Q_{tn} - I_G$ chart to identify soils with microstructure (Robertson, 2016).

Fig. 6. Abaque $Q_{tn} - I_G$ pour l'identification des sols avec microstructure (Robertson, 2016).

simplicity the relationship is shown in terms of G_0 but is intended to apply G_{ovH} .

Robertson (2009) presented contours of state parameter (ψ) for young uncemented coarse-grained (*unstructured*) soils on the normalized SBTn ($Q_{tn} - F_r$) chart and suggested that the contour for $\psi < -0.05$ could be used to separate coarse-grained 'ideal soils' that are either contractive or dilative at large strains. This was supported by case histories where flow liquefaction had occurred (Robertson, 2010). Robertson (2016) updated the CPT-based SBT charts for ideal soils based on Q_{tn} and F_r , as shown on Figure 7, to include a line, defined by $CD = 70$, that separate 'ideal soils' that are either contractive or dilative at large shear strains.

The contractive-dilative (CD) boundary can be represented by the following simplified expression:

$$CD = 70 = (Q_{tn} - 11)(1 + 0.06 F_r)^{17}. \quad (13)$$

When $CD > 70$ the soils are likely dilative at large shear strains, as shown on Figure 7. Equation (13) is a simplified fitting relationship to capture the generalized shape of the contractive-dilative boundary on the $Q_{tn} - F_r$ chart. Figure 7 also includes (as light dashed lines) the original SBTn boundaries suggested by Robertson (1990, 2009) for comparison and to retain the original grouping based on physical characteristic descriptions (e.g., sand and clay). Because Figure 7 shows behavior-based descriptions and

boundaries, it applies primarily to soils that have little or no microstructure (i.e., 'ideal soils').

Figure 7 also shows the SBTn boundaries using a modified Soil Behavior Type Index, I_B , defined as:

$$I_B = 100(Q_{tn} + 10)/(Q_{tn} F_r + 70). \quad (14)$$

The boundary represented by $I_B = 32$ represents the lower boundary for most sand-like 'ideal soils'. The boundary represented by $I_B = 22$ represents the upper boundary for most clay-like 'ideal soils'. The value of $I_B = 22$ represents the approximate boundary for a plasticity index $PI \sim 18\%$ in fine-grained 'ideal soils'.

The region represented by $22 < I_B < 32$ is defined as 'transitional soil' which are soils that can have a behavior somewhere between that of either sand-like or clay-like 'ideal soil' (e.g., low plasticity fine-grained soils, such as silt). Some 'transitional soils' can also respond in a partially drained manner during the CPT (e.g. DeJong and Randolph, 2014) and are sometimes referred to as 'intermediate soils'. The modified boundaries shown in Figure 7 are like the original boundaries from Robertson (1990) in the central part of the chart, where most young uncemented, normally consolidated 'ideal soils' plot.

The soil close to the friction sleeve of the cone has experienced very large shear strains and tends to be fully 'destructured' and remolded. Based on this observation, Lunne

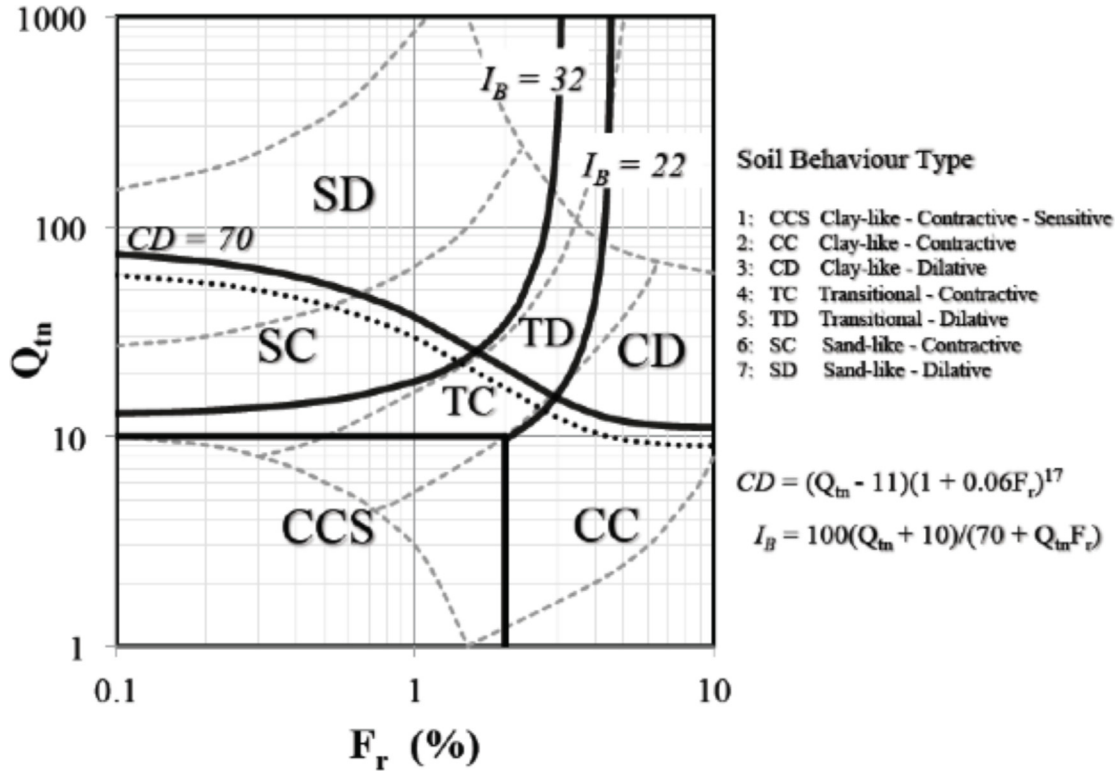


Fig. 7. SBTn chart for ideal soils based on $Q_m - F_r$ (After Robertson, 2016) (dashed lines represent SBT zones from Robertson, 1990).

Fig. 7. Abaque SBTn pour les sols idéaux basée sur $Q_m - F_r$ (d'après Robertson, 2016) (les lignes en tireté représentent les zones SBT d'après Robertson, 1990).

et al. (1997) had suggested that sensitivity (S_t) in most fine-grained 'ideal soils' could be estimated using the simplified expression:

$$S_t = 7.1/F_r. \quad (15)$$

Hence, soils with $F_r < 2\%$ tend to have a sensitivity $S_t > 3$ to 4. This boundary has been included in Figure 7 as an approximate separation between clay-like-contractive (CC) soils with moderate to low sensitivity ($S_t < 3$) from clay-like-contractive soils with higher sensitivity, $S_t > 3$ (CCS). This value of sensitivity is somewhat conservative but considered appropriate for basic classification purposes. Any boundary based on sensitivity is somewhat arbitrary but can be helpful to warn users when a soil may have higher sensitivity to disturbance with associated risk of strength loss. As shown in Figure 7 the boundary between CC and CCS is slightly more conservative than the previous boundary suggested by Robertson (1990).

The $Q_m - F_r$ chart can also be used when no pore pressure (u_2) data are available (*i.e.*, basic CPT data), since $q_c \sim q_t$ for most soils, except soft fine-grained soils ($Q_{t1} < 10$). Although the basic $Q_m - F_r$ charts tend to be more popular for most onshore projects, since they do not require accurate CPT pore pressure measurements, it can be helpful to include CPT measured pore pressures into the interpretation of soil behavior type. Figure 8 shows a modified Schneider *et al.* (2008) chart using the same behavior type terms applied in Figure 7 but using the generalized normalized cone resistance, Q_{tn} , instead of Q_{t1} . The equations to define the boundaries shown in

Figure 8 were provided by Schneider *et al.* (2008) but replacing Q_{t1} with Q_{tn} . For most clay-like soils there is little difference between Q_{tn} and Q_{t1} , since $n \sim 1.0$, but for consistency it is preferred to use Q_{tn} . Pore pressures measured in the u_2 location (just behind the cone tip) are dominated by soil behavior in shear at large strains, hence u_2 tends to reflect the behavior of the soil in shear at large strain (*i.e.*, destructured). In general, positive u_2 values tend to reflect large strain contractive behavior and negative u_2 values tend to reflect large strain dilative behavior. Hence, mostly 'contractive' behavior descriptions are shown on Figure 8 for positive values of u_2 . Based on the $Q_{tn} - F_r$ chart (Fig. 7), most fine-grained 'ideal soils' with $Q_{tn} > 12$ will have negative values of u_2 since they are generally dilative at large shear strains (*i.e.*, $YSR > 4$). However, 'structured soils' can have $Q_{tn} > 12$, due to the increased strength and stiffness, combined with large positive values of U_2 due to the loss of structure resulting in a contractive behavior at large strains. Hence, if data plot in the region represented by $Q_{tn} > 12$ combined with high positive U_2 values ($U_2 > 4$), the soils likely have significant microstructure (*i.e.*, 'structured soils') and are contractive at large shear strains. Increasing values of Q_{tn} combined with increasing positive U_2 values indicate increasing microstructure, as indicated in Figure 8. Schneider *et al.* (2008) showed that soils with increasing coefficient of consolidation (c_v) tend to show a decrease in U_2 with increasing Q_{tn} , due in part to an increase in drainage during the CPT but also an increased tendency for dilative behavior. For a given soil, increasing YSR (OCR) can be associated with increasing c_v , hence 'ideal

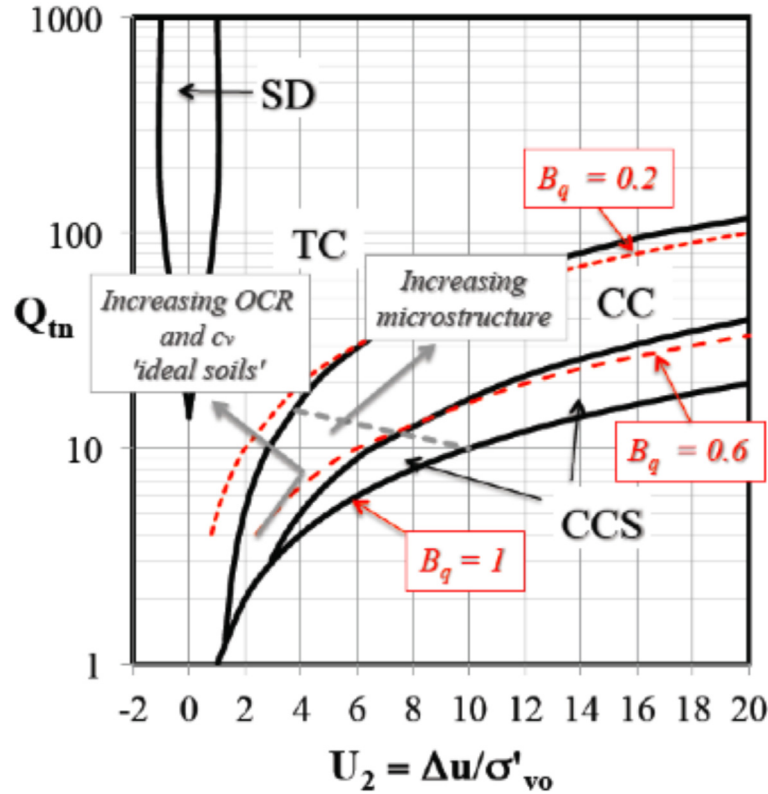


Fig. 8. Updated [Schneider et al. \(2008\)](#) chart based on $Q_{tn} - u_2$ with proposed new soil behaviour type boundaries (B_q lines in red).

Fig. 8. Abaque mis à jour de [Schneider et al. \(2008\)](#) basé sur $Q_{tn} - u_2$ avec les nouvelles limites de type de comportement du sol proposées (lignes B_q en rouge).

soil' with increasing YSR tend to show a decrease in U_2 combined with an increase in Q_{tn} , as illustrated in [Figure 8](#).

The $Q_{tn} - F_r$ chart has a modified SBTn Index I_B that can be used to define the main boundaries in soil SBTn. Likewise, the $Q_{tn} - U_2$ chart can use B_q as an approximate SBTn index to define the main boundaries, as shown in [Figure 8](#). Hence, soils with $0.2 < B_q < 0.6$ tend to be CC and soils with $0.6 < B_q < 1.0$ and $Q_{tn} > 4$ tend to be CCS, as shown in [Figure 8](#). Likewise, soils in the region defined by $U_2 > 0$ with $Q_{tn} = 20$ and $U_2 > 10$ with $Q_{tn} = 10$ appear to have significant microstructure. The combination of $Q_{tn} - F_r$ ([Fig. 7](#)) and $Q_{tn} - U_2$ ([Fig. 8](#)) can aid in identification of soils with microstructure since 'structured soils' tend to have different classification between the two charts.

A major challenge for any classification method based on CPT pore pressures is the risk that the measured pore pressures may be unreliable due to loss of saturation ([Robertson, 2009](#)). This is an issue for CPT profiles performed onshore either in soils that are dilative or when the water table is deep, and the cone is required to penetrate unsaturated and/or dilative soils for some depth.

Ideally, the above three charts ([Figs. 6–8](#)) should be used together to improve soil classification. [Figure 6](#) ($Q_{tn} - G_o/q_n$) can be used to identify soils with significant microstructure (e.g., age and/or cementation), i.e., $K^*_G > 330$, provided V_s data are available. If the soils have little or no microstructure (i.e., 'ideal soil'), [Figure 7](#) ($Q_{tn} - F_r$) should apply. [Figure 8](#) ($Q_{tn} - U_2$) can be used primarily in

fine-grained soils, when V_s data are either not available or as a supplement to [Figure 7](#), to evaluate if soils have significant microstructure and to evaluate large strain behavior, provided reliable pore pressure measurements are made. [Robertson \(2016\)](#) provided examples to illustrate the application of the three charts ([Figs. 6–8](#)) for a wide range of soils, with and without microstructure.

5 Deformation analyses

Ideally, any deformation analyses should account for the non-linear behavior of soils where the secant modulus varies with strain. The most common form to illustrate how modulus varies with strain is shown in [Figure 9](#).

The shear modulus (G) is normalized by the small strain shear modulus (G_o) that is obtained from V_s . Research (e.g., [Darendeli, 2001](#)) has shown that the rate of modulus degradation with strain is a function of the plasticity of the soil, with non-plastic sands degrading faster than clays. Also included in [Figure 9](#) are approximate ranges of shear strains for various geotechnical analyses. Most well-designed foundations have an average shear strain of less than 0.1%, whereas most stability analyses (e.g., bearing capacity) have average shear strains larger than 0.5% ([Burland, 1990](#)). Included in [Figure 9](#) are the approximate average strain levels from Ménard (pre-bored) pressuremeter tests (PMT) for either the initial loading modulus or the unload-reload modulus ([Briaud, 2013](#)).

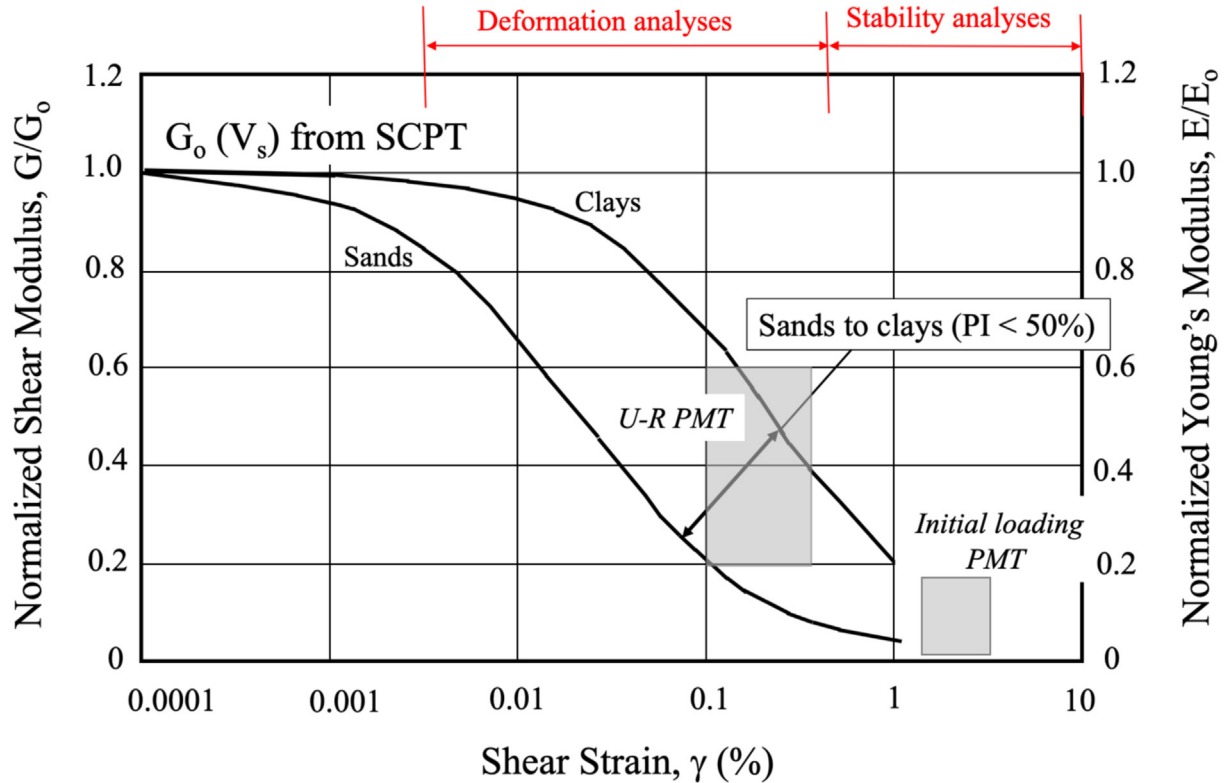


Fig. 9. Generalized variation of shear modulus as a function of shear strain.

Fig. 9. Variation normalisée du module de cisaillement en fonction de la déformation de cisaillement.

Much of the research on modulus degradation has been done on ‘ideal’ soils in the laboratory (e.g., freshly deposited sands and clays) that have no microstructure. Based on laboratory testing on ideal soils, [Fahey, \(1998\)](#) suggested that the variation of G/G_0 could be defined as a function of the degree of loading defined by:

$$G/G_0 = 1 - f(q/q_{ult})^g, \quad (16)$$

where:

- q/q_{ult} = degree of loading = $1/\text{Factor of Safety}$;
- q = applied stress/load;
- q_{ult} = failure/ultimate stress/load;
- f and g are curve fitting parameters.

[Fahey \(1998\)](#) and [Mayne \(2005\)](#) suggested that values of $f=1$ and $g=0.3$ are appropriate for soils that have no microstructure. The degree of loading is essentially the inverse of the Factor of Safety for any design application (e.g., bearing capacity for shallow and deep foundations). The ultimate load (e.g., bearing capacity) can be estimated from the CPT tip resistance, since it is a measure of the large strain strength of the soil (e.g., [Robertson and Cabal, 2022](#)). Hence, for ‘ideal’ soils with little or no microstructure, it is possible to estimate the variation of modulus as a function of degree of loading based on *in situ* V_s (G_0) from the SCPT. [Mayne \(2005\)](#) showed very good agreement between stress strain curves estimated from the SCPTu and laboratory measured results on undisturbed samples. This approach was extended to estimate the non-linear load-displacement response of shallow

([Mayne, 2005](#)) and deep foundations ([Mayne and Woeller, 2008](#)). This approach may not apply so well for soils that have some microstructure, where independent measurements of both the small and large strain stiffness can be used to better define the variation of modulus with strain.

[Figure 9](#) illustrates the potential application of combining *in situ* test data that uses the small strain modulus (G_0) from V_s data (e.g., using the SCPT) with the larger strain modulus from the PMT. The PMT can measure the modulus at both intermediate strains (unload-reload) and relatively large strains (initial loading) and when combined with the small strain modulus can be used to essentially define the full stress-strain curve over a wide range of strains. [Figure 9](#) illustrates the need to control and measure the strain level over which the modulus is determined (e.g., unload-reload loops in a PMT). The estimation of the non-linear stress strain relationship has applications for a wide range of deformation problems, including non-linear numerical analyses.

6 Links to Pressuremeter Test (PMT)

The Ménard (pre-bored) pressuremeter test (PMT) is popular in France and many other countries. It involves pre-drilling a hole and expanding a cylindrical membrane to record the pressure and radial expansion of the membrane against the soil. The main parameters recorded from a PMT are the limit pressure, P_L (i.e., pressure to expand the membrane to twice its initial volume) and the initial loading modulus, sometimes referred to as the Ménard modulus (E_M). Increasingly the

unload-reload modulus (E_{UR}) is also measured from small unload-reload loops.

At small strains, the shear modulus G_o is related to Young's Modulus, E_o , through elasticity theory:

$$E_o = 2G_o(1 + \nu), \quad (17)$$

where: ν = Poisson's ratio.

In the PMT, it is common to calculate Ménard's Modulus, E_{PMT} , using equation (17) with an assumed average $\nu = 0.33$. Hence:

$$E_{PMT} = 2.66G_{PMT}, \quad (18)$$

where:

- E_{PMT} is either E_M or E_{UR} ;
- G_{PMT} is either G_M or G_{UR} .

Hence, the modulus reduction curves shown in Figure 9 can also represent E/E_o .

Briaud (2013) and others have shown that the average strain around the pressuremeter is approximately 30% of the measured strain and that the typical measured strain for the initial loading is between 3 to 5%, depending on soil conditions. The typical strain for unload-reload loops varies from 0.1 to 0.5%, depending on the soil stiffness and the amount of unloading allowed.

The main challenge with E_{UR} is that, unlike E_M , it is not precisely defined and depends on the strain amplitude over which the loop is performed and to a lesser extent on the stress level at which the loop is performed. However, experience has shown that the initial loading modulus is influenced by borehole disturbance.

Figure 9 summarizes the link between the small strain modulus, either G_o or E_o , obtained from V_s , and the larger strain moduli measured from the PMT. With careful control and measurement of the strain range, the unload-reload modulus, E_{UR} (or G_{UR}) can be compared with G_o (or E_o) to provide an estimate of the variation of modulus with strain. Likewise for E_M (or G_M), but since the strain level is high and borehole disturbance more varied, the link maybe less helpful.

The limit pressure is essentially a measure of the shear strength of the soil and can be linked to the CPT tip resistance. Unfortunately, the limit pressure is not always a true limit pressure because failure is not always reached when the membrane has expanded by twice the initial volume, especially in stronger soils. Hence, the link to the CPT tip resistance is not always reliable, especially in stronger soils (e.g., dense sands and stiff over consolidated clays). For soils with microstructure, both the CPT q_t and PMT P_L are predominately large strain measurements and represent a response when most of the microstructure has been destroyed. Ideally, any links between CPT q_t and PMT P_L should be done using normalized parameters to remove the influence of overburden stress. Hence, the correlation will be done using the net limit pressure, P_L^* defined by:

$$P_L^* = P_L - \sigma_{ho}, \quad (19)$$

where σ_{ho} is the *in situ* total horizontal stress and the normalized net limit pressure is P_L^*/σ'_{vo} .

For soft clay-like soils the net limit pressure is linked to the undrained shear strength ratio, s_u/σ'_{vo} by the following based on Gibson and Anderson, (1961):

$$P_L^*/\sigma'_{vo} \sim 5.5 s_u/\sigma'_{vo}. \quad (20)$$

Likewise, s_u/σ'_{vo} is linked to the normalized cone resistance, Q_{tn} by:

$$s_u/\sigma'_{vo} = Q_{tn}/N_{kt}, \quad (21)$$

where N_{kt} is the cone factor that has an average value of 14.

Hence, in clay-like soils, the link between PMT and CPT is:

$$P_L^*/\sigma'_{vo} \sim 0.4 Q_{tn}. \quad (22)$$

In clay-like soils, the YSR (or OCR) can be estimated using the following:

$$YSR = 0.33 Q_{tn}. \quad (23)$$

When $Q_{tn} \sim 12$, the $YSR \sim 4$ and $P_L^*/\sigma'_{vo} \sim 5$.

In sand-like soils, Yu *et al.* (1996) used theoretical solutions to link *in situ* state parameter, ψ , to q_t/P_L and showed that when $\psi = -0.05$, $q_t/P_L = 5$ increasing to 10 when $\psi = -0.20$.

Based on the above discussion, Figure 10 shows the approximate contours of P_L^*/σ'_{vo} on the normalized CPT-based SBT chart. The contour for $P_L^*/\sigma'_{vo} = 5$ is like the contractive-dilative boundary on the CPT-based SBT chart. For clay-like soils, $P_L^*/\sigma'_{vo} = 5$, implies YSR (OCR) of around 4, which is also considered the boundary between contractive and dilative behavior at large strains. Hence, $P_L^*/\sigma'_{vo} = 5$ is a reasonable boundary to separate contractive and dilative behavior at large strains and the normalized parameter P_L^*/σ'_{vo} appears to be linked to *in situ* state (either ψ or OCR).

Briaud (2013) and Bustamante and Frank (1999) gave typical comparisons between PMT parameters and CPT for a wide range of soils. The contours on Figure 10 are generally consistent with these published links between PMT and CPT.

Based on the above discussion there is the possibility to identify microstructure by combining seismic methods (either SCPT or MASW) to obtain G_o and PMT P_L^* , since G_o is a small strain measurement and P_L^* is a larger strain measurement. A possible relationship could exist between G_o/P_L^* as a function of P_L^*/σ'_{vo} that could be used to identify microstructure, but a suitable database is not available to the author to evaluate this concept.

7 Summary

Significant developments have been made in the use and application of SCPTu since it was first developed about 40 years ago. The SCPTu can provide 7 or more measurements in one, cost effective *in situ* test to produce near continuous profiles of the following measurements: $q_t, f_s, u, V_s, V_p, t_{50}, u_o, i$. No other *in situ* test provides this amount of measurement in such a cost-effective manner.

The important role of soil microstructure is increasingly becoming recognized and the SCPTu has the potential to

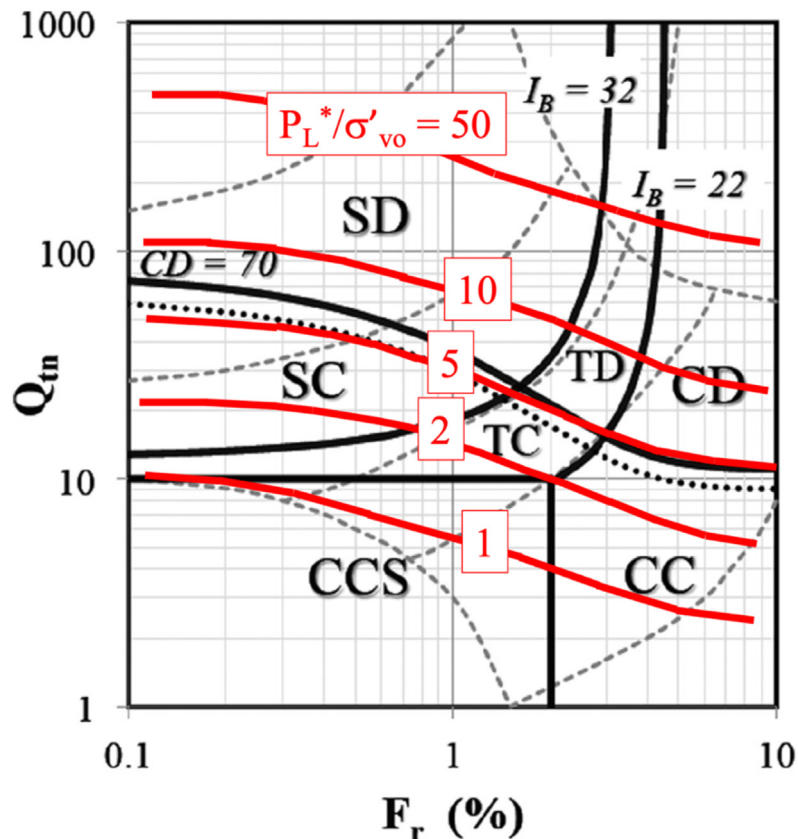


Fig. 10. Approximate contours of PMT Normalized Net Limit Pressure, P_L^*/σ'_{vo} , on the CPT SBT chart by Robertson (2016).

Fig. 10. Contours approximatifs de la pression limite nette normalisée au PMT, P_L^*/σ'_{vo} , sur l'abaque CPT SBT de Robertson (2016).

identify soil microstructure. Likewise, the importance of non-linearity in soil stiffness is also becoming recognized and the SCPTu proves a measure of the fundamental small strain stiffness, G_0 . Based on these developments the SCPTu is becoming the primary *in situ* test around the world for most soils.

Given the popularity of the pre-bored (Ménard) pressuremeter test (PMT) in France, some links between SCPTu results and PMT results are suggested. The SCPTu is ideal for relatively soft soils compared to the PMT that is better suited to soils, where penetration with the SCPTu maybe limited. The combination of surface geophysics, such as the MASW, and either SCPTu or PMT has the potential to provide insights into soil microstructure and non-linear soil stiffness.

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