

Cone penetration test (CPT)-based soil behaviour type (SBT) classification system — an update

P.K. Robertson

Abstract: A soil classification system is used to group soils according to shared qualities or characteristics based on simple cost-effective tests. The most common soil classification systems used in geotechnical engineering are based on physical (textural) characteristics such as grain size and plasticity. Ideally, geotechnical engineers would also like to classify soils based on behaviour characteristics that have a strong link to fundamental in situ behaviour. However, existing textural-based classification systems have a weak link to in situ behaviour, since they are measured on disturbed and remolded samples. The cone penetration test (CPT) has been gaining in popularity for site investigations due to the cost-effective, rapid, continuous, and reliable measurements. The most common CPT-based classification systems are based on behaviour characteristics and are often referred to as a soil behaviour type (SBT) classification. However, some confusion exists, since most CPT-based SBT classification systems use textural-based descriptions, such as sand and clay. This paper presents an update of popular CPT-based SBT classification systems to use behaviour-based descriptions. The update includes a method to identify the existence of microstructure in soils, and examples are used to illustrate the advantages and limitations of such a system.

Key words: cone penetration test (CPT), soil classification, microstructure, case histories.

Résumé : Un système de classification des sols est utilisé pour grouper des sols selon les qualités partagées ou les caractéristiques basées sur des essais efficaces peu coûteux. Les systèmes de classification de sol les plus courants utilisés en génie géotechnique sont basés sur des caractéristiques physiques (textures) tels que la taille des grains et la plasticité. Idéalement, les ingénieurs géotechniques tiennent également à classer les sols en fonction des caractéristiques comportementales qui ont un lien étroit aux comportements fondamentaux in situ. Cependant, les systèmes existants de classification basés sur la texture ont un maillon faible au comportement in situ, car ils sont mesurés sur des échantillons perturbés et remaniés. Les essais faits au moyen de pénétration au cône (« cone penetration test » CPT) ont gagné en popularité pour les études de sites en raison des mesures rentables, rapides, continues et fiables. Les systèmes de classification basés sur le CPT les plus courants sont basés sur les caractéristiques du comportement et sont souvent appelés une classification de type de comportement de sol (« soil behaviour type » SBT). Toutefois, une certaine confusion existe puisque la plupart de systèmes de classification SBT basés sur le CPT utilisent des descriptions axées sur la texture, comme le sable et l'argile. Cet article présente une mise à jour des systèmes populaires de classification SBT basés sur le CPT, afin d'utiliser les descriptions basées sur le comportement. La mise à jour inclut une méthode pour identifier l'existence de microstructures dans les sols et les exemples servent à illustrer les avantages et les limites d'un tel système. [Traduit par la Rédaction]

Mots-clés : essai de pénétration au cône (CPT), classification des sols, microstructure, historiques de cas.

Introduction

A soil classification system is used to group soils according to shared qualities or characteristics based on simple cost-effective tests. The most common soil classification systems used in geotechnical engineering are based on physical (textural) characteristics such as grain size and plasticity (e.g., Unified Soil Classification System, USCS). These textural-based classification systems have been used for over 70 years to provide general guidance through empirical correlations based on past field experience. Unfortunately, empirical correlations between simple physical index properties measured on remolded samples and in situ soil behaviour have significant uncertainty. Ideally, geotechnical engineers should also classify soils based on fundamental behaviour characteristics that have a strong link to in situ behaviour. A combined classification based on both physical and behaviour characteristics would be very helpful for many geotechnical projects.

The cone penetration test (CPT) has been gaining in popularity for site investigations due to the cost-effective, rapid, continuous,

and reliable measurements. The most common CPT-based classification systems are based on behaviour characteristics and are often referred to as a soil behaviour type (SBT) classification (e.g., Robertson 1990). However, most CPT-based SBT classification systems use textural-based descriptions, such as sand and clay, that can cause some confusion in geotechnical practice. The objective of this paper is to present an update to the Robertson (1990, 2009) and Schneider et al. (2008) CPT-based SBT classification system with behaviour-based descriptions for each soil group. The importance of microstructure and how it can influence CPT-based classification is also discussed. The paper will also attempt to collate, update, and summarize the growing experience that exists to guide in the classification of soils based on CPT measurements.

Soil classification

The most common soil classification system used by engineers and geologists in North America is the USCS (ASTM 2011, D2487-11). The system, similar to others used around the world, is based

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on physical characteristics of grain-size distribution and plasticity (Atterberg limits) measured on disturbed and remolded samples. The USCS groups soils into two broad groups: coarse-grained (sand and gravel) and fine-grained (silt and clay). Coarse-grained soils are those with more than 50% retained on or above the No. 200 sieve (>0.075 mm) and are further grouped based on grain size and fines content (e.g., silty sand). Fine-grained soils are those with 50% or more passing the No. 200 sieve and are further grouped based on plasticity (high or low).

Although the USCS is based on physical characteristics, past field experience has shown that approximate generalized behaviour characteristics of the main groups can be described as follows:

Coarse-grained soils tend to have

- High strength,
- Low compressibility, and
- High permeability.

Fine-grained soils tend to have

- Medium to low strength,
- Medium to high compressibility, and
- Low permeability.

The actual in situ soil behaviour depends on many other factors such as geologic processes related to origin, environmental factors (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 behaviour is strongly influenced by geologic factors such as age and cementation. Typically, the link is most successful when applied to young (Holocene- and Pleistocene-age), uncemented silica-based deposits with limited stress and strain history (e.g., older heavily overconsolidated clays can have similar strength and compressibility characteristics to younger uncemented sands).

Generalized soil behaviour

The behaviour of natural soils is complex, with many extensive publications attempting to describe this behaviour (e.g., [Leroueil and Hight 2003](#)). The following is a very short summary of some key features of soil behaviour as it relates to a possible behaviour type classification system.

The following features describe the essential elements of soil behaviour (e.g., [Atkinson 2007](#)):

- Soil can change in volume due to rearrangement of grains and void space changes.
- Soil is essentially frictional where strength and stiffness increases with normal stress and with depth in the ground.
- Soil is essentially inelastic where response is nonlinear to loading beyond an initial very small threshold strain.

As stated previously, in situ soil behaviour depends on many factors such as geologic processes related to origin (e.g., depositional and compositional features), environmental factors (e.g., stress and temperature), as well as physical and chemical processes (e.g., aging and cementation). The powerful early concepts of critical-state soil mechanics were based on tests performed on isotropically consolidated reconstituted (clay) samples and can be representative of saturated “ideal soils” ([Leroueil and Hight 2003](#)). Many natural soils have some form of structure that can make their in situ behaviour 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 (see [Fig. 1](#)) the main differences in mechanical behaviour between soils with microstructure (i.e., “structured soils”) and ideal soils (i.e., unstructured soils). Compared to 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, 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.

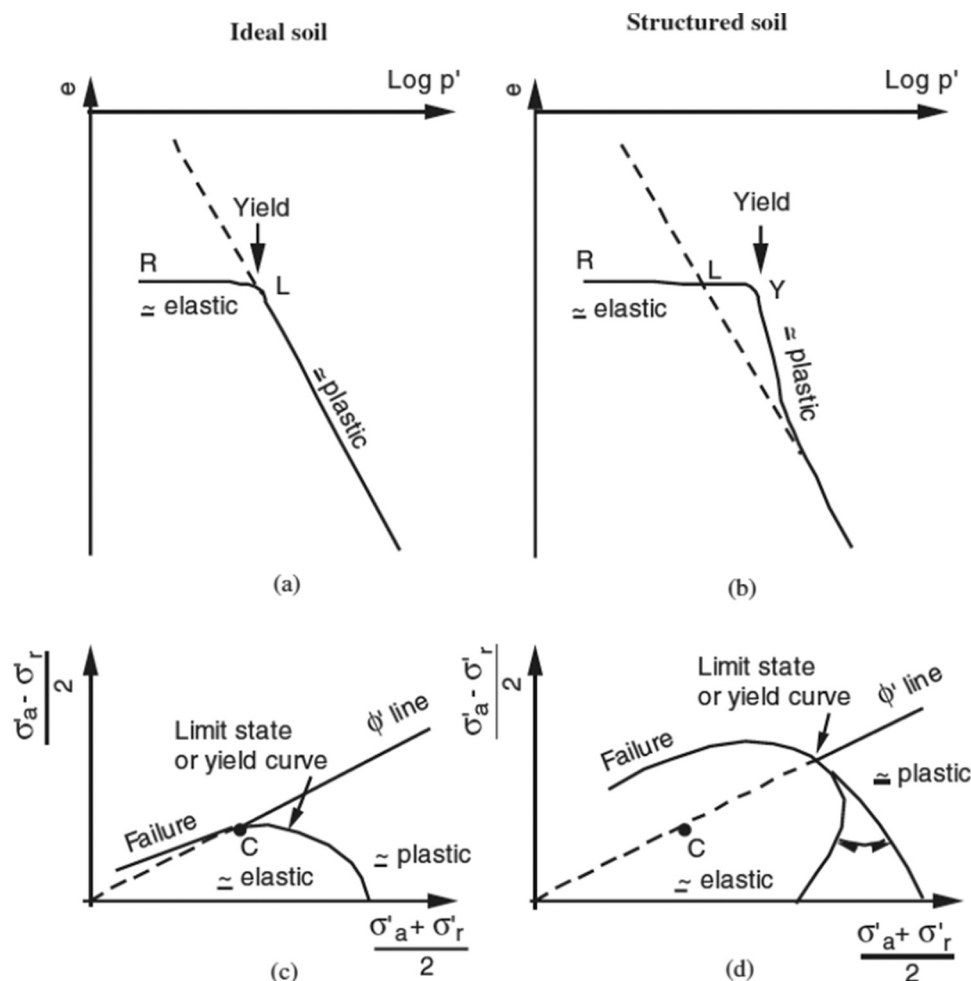
Critical-state soil mechanics is based on the observation that soils ultimately reach critical state at large strains, and at critical state there is a unique relationship between shear stress, normal effective stress, and void ratio. Since critical state is independent of the initial state, the parameters 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 a number of ways. In fine-grained soils, it is common to define the current state in terms of overconsolidation ratio (OCR) that is related to the normal compression line, since fine-grained ideal soils tend to have a unique normal compression line that is essentially parallel to the critical-state line. 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 critical-state line, since the normal compression line is not unique (e.g., [Been and Jefferies 1985](#)). At low confining stress, the critical-state line for many clean silica-based sands can be very flat in terms of void ratio versus log mean effective stress; 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 important difference between the behaviour of ideal soils that are either “loose” or “dense” of critical state. Soils that are loose of critical state tend to contract on drained loading (or where pore pressures rise on undrained shear). Soils that are dense of critical state 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 behaviour.

The behaviour 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 behaviour characteristic for many geotechnical problems.

[Jefferies and Been \(2006\)](#) had suggested that coarse-grained ideal soils with a state parameter $\psi < -0.05$ will tend to dilate at large strains when loaded in drained shear. Hence, coarse-grained

Fig. 1. Schematic behaviour of ideal and structured soils (after Leroueil 1992). C, critical state; e , void ratio; p' , mean effective stress; R, overconsolidated stress state; L, yield stress state for ideal soil; Y, yield stress state for structured soil; σ'_a , axial effective stress; σ'_r , radial effective stress; ϕ' , friction angle.



saturated ideal soils with $\psi < -0.05$ will tend to generate a drop in pore pressure and increase in effective stress at larger strains in undrained shear and tend to be strain hardening. Likewise, fine-grained saturated ideal soils with an OCR > 4 tend to dilate when loaded in drained shear and also tend to generate a drop in pore pressure in undrained shear. Most soils tend to contract at small strains (i.e., develop positive pore pressures under undrained loading), which is a major feature in soils that experience cyclic liquefaction. Clearly classifying soils by their tendency to dilate or contract at large strains under shearing is an important behaviour characteristic, since it can guide the engineer in terms of appropriate behaviour parameters and critical loading conditions.

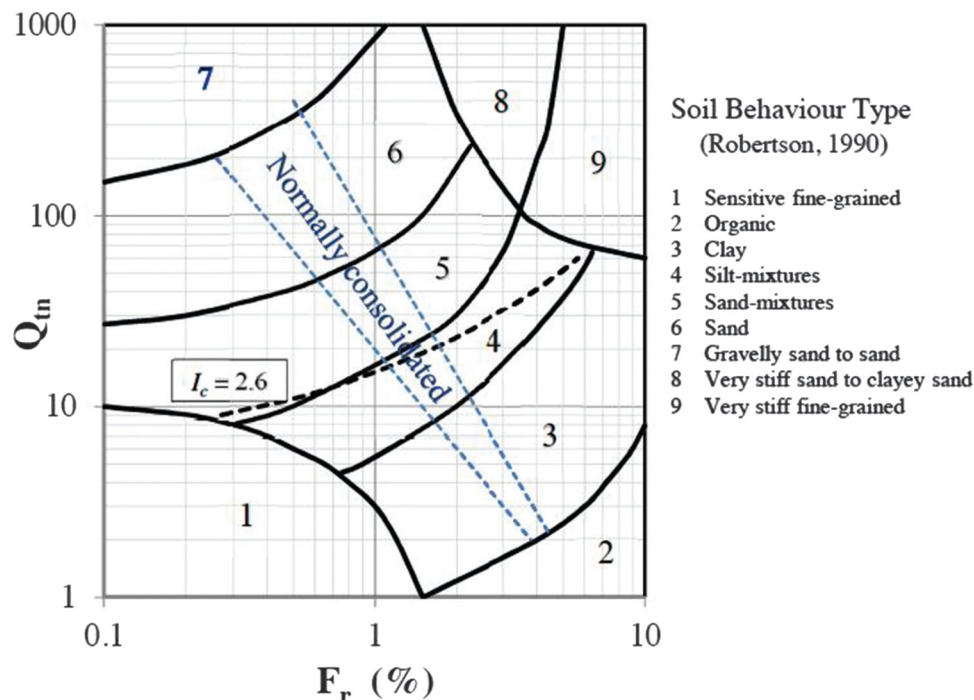
Idriss and Boulanger (2008) classified soils as either sandlike or claylike in their behaviour, where sandlike soils are susceptible to cyclic liquefaction and claylike soils are not susceptible to cyclic liquefaction. Idriss and Boulanger (2008) also suggested that fine-grained soils transition from behaviour that is more fundamentally like sands to behaviour that is more fundamentally like clays over a fairly narrow range of plasticity index (PI). Sandlike soils tend to have $PI < 10\%$ and claylike soils tend to have $PI > 18\%$ (Bray and Sancio 2006). The transition from more sandlike to more claylike behaviour is conceptually similar to the transition in the USCS from coarse-grained (nonplastic) to fine-grained (plastic) soils, although some low-plastic or nonplastic fine-grained soils (e.g., silt and low-plasticity clay) can behave more sandlike, as suggested by Idriss and Boulanger (2008). Likewise, the transition

from saturated soils that are more sandlike to more claylike 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.

A modified classification system based on behaviour characteristics can be built around groupings that divide soils that show either dilative or contractive behaviour at large strains and soils that are predominately more sandlike or more claylike. There can also be a group that captures soils that are in transition from more sandlike to more claylike, since the change occurs over a range of behaviour type.

Since natural soil behaviour is complex, any classification system based on behaviour characteristics should involve multiple measurements that are repeatable and capture different aspects of in situ soil behaviour. For classification systems to be effective and easy to use, they must also be based on rather simple, cost-effective repeatable tests. For an in situ test to meet these demands, the test should be simple, cost effective, and provide several repeatable independent measurements. One of the most popular modern in situ tests that is applicable to uncemented soil is the CPT. The CPT is fast (20 mm/s), cost effective, and provides continuous and repeatable measurements of several parameters. The basic CPT records tip resistance (q_c) and sleeve resistance (f_s). The CPTu provides the addition of penetration pore pressure (u), often in the u_2 location just behind the tip, combined with a

Fig. 2. CPT-based SBTn chart suggested by Robertson (1990) and updated by Robertson (2009). [Colour online.]



measure of the rate of pore-pressure dissipation during a pause in the penetration process, often expressed as the time required to dissipate 50% of the excess pore pressure (t_{50}). The CPTu can also provide in situ equilibrium pore pressure after 100% dissipation (u_0), which is helpful to define the in situ piezometric profile at the time of the CPTu. The seismic CPTu (SCPTu) provides the additional measurement of in situ shear wave velocity (V_s) and, in some conditions, in situ compression wave velocity (V_p). Hence, the SCPTu can provide up to seven independent measurements in one cost-effective test. Ideally, a classification system should include all these measurements to be fully effective. However, a practical classification system can still be effective based on either two or three measurements, provided limits are placed on the range of applicable soils (e.g., restricted to predominately ideal soils).

Any behaviour-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 behaviour similar to most ideal soils, i.e., has little or no microstructure. It will be shown that the SCPTu measurements have the potential to identify soils with significant microstructure.

Existing CPT-based classification

One of the major applications of the CPT has been the determination of soil stratigraphy and the identification of soil type. This has been accomplished using charts that link cone measurements to soil type. The early charts developed in the Netherlands were based on measured cone resistance, q_c , and sleeve resistance, f_s , using a mechanical cone (e.g., Begemann, 1965) and showed that there is an approximate linear link between q_c and f_s for a given soil type. Early charts using q_c and friction ratio ($R_f = f_s/q_c$ in percent) were proposed by Douglas and Olsen (1981), but the charts proposed by Robertson et al. (1986) and Robertson (1990, 2009) have become very popular. Robertson et al. (1986) and Robertson (1990, 2009) stressed that the charts were predictive of soil behaviour type (SBT), since the cone responds to the in situ mechanical behaviour of the soil (e.g., strength, stiffness, compressibility, and

drainage) and not directly to classification criteria based on physical characteristics, such as grain-size distribution and soil plasticity.

The CPT-based normalized soil behaviour type (SBTn) method suggested by Robertson (1990) was based on the following normalized parameters:

- (1) $Q_t = (q_t - \sigma_{vo})/\sigma'_{vo}$
- (2) $F_r = [f_s/(q_t - \sigma_{vo})] 100\%$
- (3) $B_q = (u_2 - u_0)/(q_t - \sigma_{vo}) = \Delta u/(q_t - \sigma_{vo})$

where q_t is the cone resistance corrected for water effects, where $q_t = q_c + u_2(1 - a)$; a is the cone area ratio, typically around 0.8; σ_{vo} is the current in situ total vertical stress; σ'_{vo} is the current in situ effective vertical stress; u_2 is the penetration pore pressure (immediately behind cone tip); u_0 is the current in situ equilibrium water pressure; and Δu is the excess penetration pore pressure ($= u_2 - u_0$).

Robertson (1990) suggested two charts based on either Q_t - F_r and Q_t - B_q but recommended that the Q_t - F_r chart (illustrated in Fig. 2) was generally more reliable, since the CPT penetration pore pressures (u_2) can suffer from lack of repeatability due to loss of saturation, especially when performed onshore at locations where the water table is deep and (or) in very stiff soils. The sleeve resistance (f_s) is often considered less reliable than the cone resistance (q_c) due to variations in cone design (e.g., Lunne et al. 1986). However, Boggess and Robertson (2010) provided recommendations on methods to improve the repeatability and reliability of sleeve resistance measurements by using cone designs with separate load cells, equal end-area sleeves, attention to tolerance requirements, and careful test procedures. Robertson (2009) also showed that, in softer soils, the SBTn charts are not overly sensitive to variations in f_s . The chart shown in Fig. 2 is based on the corrected cone resistance (q_t) that requires pore-pressure measurements to make the correction. However, the difference between q_c and q_t is generally small, except in very soft fine-grained soils. Hence, the chart in Fig. 2 is often used successfully with the basic CPT data of

q_c and f_s in most soils (i.e., q_c used in eq. (1)). Since soils are essentially frictional and both strength and stiffness increase with depth, normalized parameters are more consistent with in situ soil behaviour. The chart in Fig. 2 is often referred to as the Robertson SBTn chart.

Since 1990, there have been other CPT soil behaviour type charts developed (e.g., Jefferies and Davies 1993; Olsen and Mitchell 1995; Eslami and Fellenius 1997; Ramsey 2002; Schneider et al. 2008, 2012). Each chart tends to have advantages and limitations, some of which were briefly discussed by Robertson (2009). A common feature in many of these CPT-based methods is that the classification system uses groupings based on traditional physical descriptions (e.g., sand and clay) even though the methods are based on behaviour measurements (e.g., either q_c or q_t and f_s). This has resulted in some confusion in geotechnical practice.

Jefferies and Davies (1993) identified that a soil behaviour type index, I_c , could represent the SBTn zones in the Q_t - F_r chart where I_c is essentially the radius of concentric circles that define the boundaries of soil type. Robertson and Wride (1998) modified the definition of I_c to apply to the Robertson (1990) Q_t - F_r chart, as defined by

$$(4) \quad I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$$

The contours of I_c can be used to approximate the SBTn boundaries. The circular shape of the I_c boundaries provides a reasonable fit to the SBTn boundaries in the center of the chart where much of the data exists for most normally to lightly overconsolidated ideal soils. For some soils, the circular shape is a less effective fit to the original SBT boundaries, as illustrated in Fig. 2 for $I_c = 2.6$ and discussed by Robertson (2009). Robertson and Wride (1998) had suggested that $I_c = 2.6$ was an approximate boundary between soils that were either more sandlike or more claylike, based on cyclic liquefaction case histories that were limited to predominately silica-based ideal soils that were essentially normally consolidated. However, experience has shown that the $I_c = 2.6$ boundary is not always effective in soils with significant microstructure.

Robertson (2009) updated the normalized cone resistance and the associated SBTn chart, using a normalization with a variable stress exponent, n , where

$$(5) \quad Q_{tn} = [(q_t - \sigma_v)/p_a](p_a/\sigma'_{vo})^n$$

where $(q_t - \sigma_v)/p_a$ is the dimensionless net cone resistance; $(p_a/\sigma'_{vo})^n$ is the stress normalization factor; p_a is the atmospheric reference pressure in the same units as q_t and σ_v ; and n is the stress exponent that varies with SBTn, and defined by

$$(6) \quad n = 0.381(I_c) + 0.05(\sigma'_{vo}/p_a) - 0.15$$

where $n \leq 1.0$.

The chart shown in Fig. 2 uses Q_{tn} , instead of the original Q_t suggested by Robertson (1990), and where I_c is also determined using Q_{tn} (Robertson 2009). In most fine-grained soils, when $I_c > 2.6$, $Q_t \sim Q_{tn}$, since $n \sim 1.0$. Likewise, when $\sigma'_{vo} = 1$ atm (100 kPa) and $\sigma'_{vo} > 10$ atm (1 MPa), $Q_t = Q_{tn}$. The largest difference between Q_t and Q_{tn} occurs in coarse-grained soils at shallow depth ($\sigma'_{vo} < 1$ atm), when $Q_t > Q_{tn}$, (since $n < 1.0$). Jefferies and Been (2006) had suggested that the stress exponent in eq. (5) should always be $n = 1.0$. However, due to the nonlinear variation of shear stiffness (G) with depth, the effective stress exponent in coarse-grained soil can be less than 1.0, as indicated by eq. (6).

The original method suggested by Robertson (1990) included a chart based on Q_t and B_q . However, Schneider et al. (2008) showed

that B_q may not be the best form of normalized CPT pore pressure to identify soil type and suggested a chart based on Q_t and U_2 (where $U_2 = \Delta u_2/\sigma'_{vo}$). The Schneider et al. (2008) pore-pressure chart applies mostly to claylike soils, since it requires a measured excess penetration pore pressure (Δu_2) and was developed primarily to aid in separating whether CPT penetration is drained, undrained, or partially drained. The Schneider et al. (2008) Q_t - U_2 chart uses slightly different grouping of soil type (and description terms) compared to the Robertson (1990) chart, which can also lead to some confusion when applying both.

There is now more than 25 years of experience using the Robertson SBTn chart, and in general, the chart provides good agreement between USCS-based classification and CPT-based SBTn (e.g., Molle, 2005) for most soils with little microstructure. Robertson (2009) discussed several examples where there can be observed differences. In summary, the Robertson SBTn chart tends to work well in ideal soils (i.e., unstructured soils) but can be less effective in structured soils. Schneider et al. (2012) suggested adjusting the boundaries to be more hyperbolic in shape so that they are steeper in the region of higher F_r values. Based on the accumulated experience using the SBTn chart, it is possible to update the chart and to modify the descriptions of "soil type" to use terms based more on soil behaviour. It is also possible to identify soils with significant microstructure using additional CPT-based measurements (e.g., SCPT).

Proposed modified SBTn charts

As discussed in the section on soil behaviour, 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 behaviour 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.

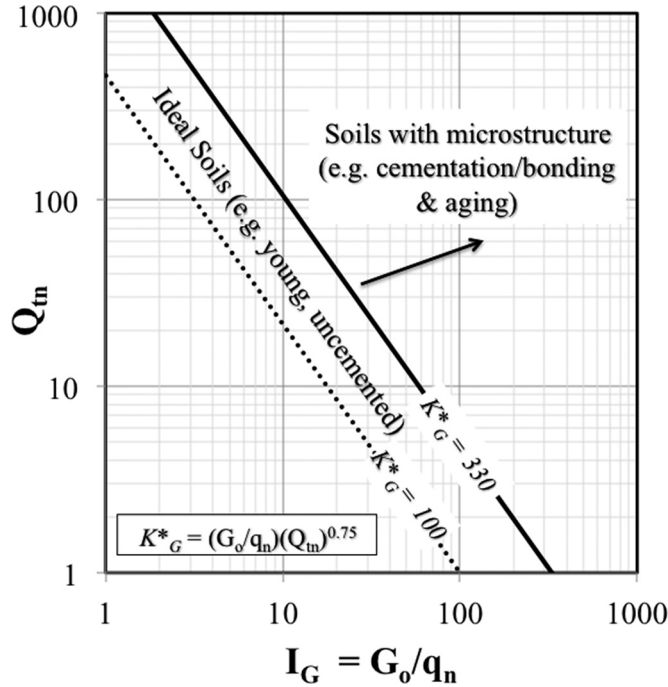
The combined information from the SCPT has the potential to aid in identification of possible microstructure in soils to either supplement existing geologic information or when geologic information maybe either lacking or uncertain. Eslamizaad and Robertson (1996) and Schnaid (2009) suggested that the 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) extended the link between CPT and G_o to establish a method to identify sandy soils with microstructure. Schneider and Moss (2011) suggested using an empirical parameter, K_G , defined in eq. (7) (modified from Rix and Stokoe 1991):

$$(7) \quad K_G = (G_o/q_t)(Q_{tn})^{0.75}$$

where G_o has the same units as q_t , and Q_{tn} is dimensionless; G_o is the small-strain shear modulus ($= \rho(V_s)^2$); V_s is the shear wave velocity; ρ is the soil mass density ($= \gamma/g$); γ is the soil unit weight; and g is the acceleration due to gravity.

Schnaid (2009) and Robertson (2009) proposed similar relationships but with slightly different exponents for Q_{tn} . The ratio G_o/q_t is essentially a small-strain rigidity index (I_c), 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} . The works by Eslamizaad and Robertson (1996),

Fig. 3. Proposed Q_{tn} - I_G chart to identify soils with microstructure.



Schnaid (2009), and Schneider and Moss (2011) were focused primarily on coarse-grained soils. Robertson (2009) 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

$$(8) \quad I_G = G_o/q_n$$

where $q_n = (q_t - \sigma_v)$.

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

$$(9) \quad K_G^* = (G_o/q_n)(Q_{tn})^{0.75}$$

Figure 3 presents a plot of Q_{tn} versus I_G similar to that shown by Schneider and Moss (2011) but extended to cover a wider range of soils. Schneider and Moss (2011) showed that most young, uncemented ("ideal") silica-based sands have $100 < K_G < 330$, with an average value of around 215. In most coarse-grained soils, $q_t \sim q_n$, since $q_t > \sigma_v$; hence, the data presented by Schneider and Moss (2011) in terms of K_G also plot in the same region in terms of K_G^* on the modified plot shown in Fig. 3. Since Fig. 3 has been extended to include soft clays, the modification to use G_o/q_n becomes important, since q_n is a direct measure of soil strength in claylike soils and can be significantly smaller than q_t in soft clay.

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 young and uncemented (i.e., have little or no microstructure) and can be classified as ideal soils (unstructured) where many 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

behaviour is often gradual, and any separating criteria can be somewhat arbitrary. Data suggests that very young uncemented soils tend to have K_G^* values closer to 100, whereas soils with some microstructure (e.g., older deposits) tend to have K_G^* values closer to 330. As will be shown later, 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 behaviour.

A challenge when calculating K_G^* , which will be illustrated later, 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 10–50 mm depth intervals, whereas V_s (and hence G_o) is typically measured over 500–1000 mm (or larger) depth intervals. Hence, 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. In the examples shown later, the CPT data (q_n) was averaged over the depth interval that was used to obtain the V_s measurement (e.g., if V_s was taken at 1 m intervals, the associated q_n value was taken as the average value over the same 1 m interval). Hence, the calculated value of K_G^* can show some variability in nonhomogenous 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 Fig. 3 is based on G_{ovH} (and V_{svH}) that is measured primarily using the SCPT. For simplicity, the relationship is shown in terms of G_o 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).

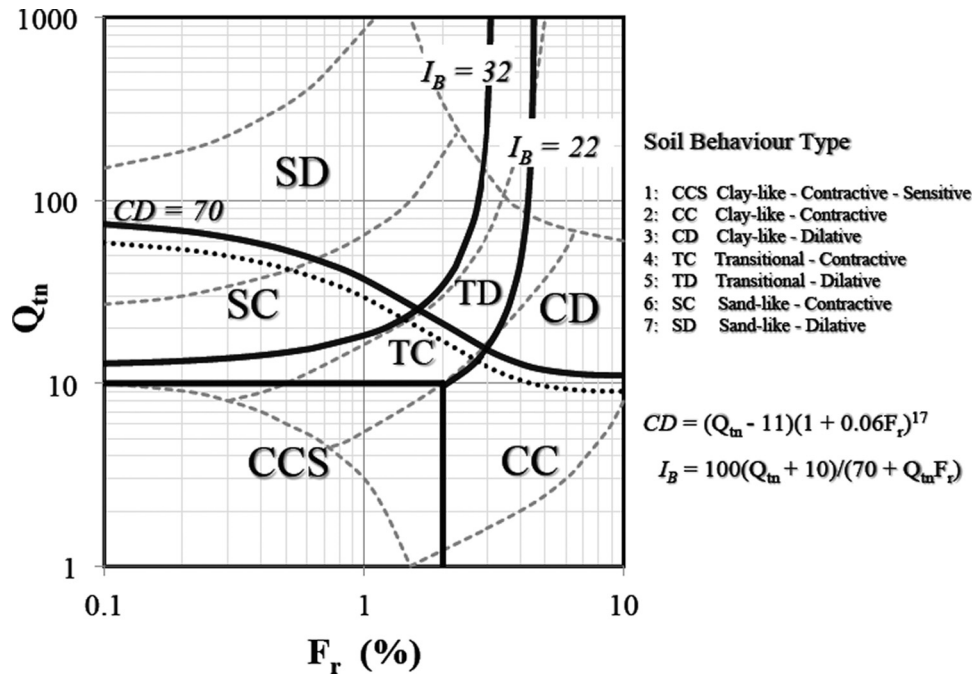
Robertson (2009), Mayne (2014), and others have shown that most fine-grained ideal soils with an OCR > 4 should have $Q_{tn} > 12$ and are predominately dilative at large shear strains. Hence, combining these two criteria, it is possible to develop a simple $Q_{tn}-F_r$ based boundary that would separate ideal soils that are either contractive or dilative at large shear strains, as shown in Fig. 4 by the solid line (marked CD = 70). The contractive–dilative (CD) boundary can be represented by the following simplified expression:

$$(10) \quad CD = 70 = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$$

When $CD > 70$, the soils are likely dilative at large shear strains, as shown in Fig. 4. Equation (10) is a simplified fitting relationship to capture the generalized shape of the contractive–dilative boundary on the $Q_{tn}-F_r$ chart. Figure 4 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 Fig. 4 shows behaviour-based descriptions and boundaries, it applies primarily to soils that have little or no microstructure (i.e., ideal soils).

A CPT-based boundary between contractive and dilative soils depends on many variables (e.g., in situ stress state, soil plastic hardening), and there is a transition between ideal soils that are predominately contractive to soils that are predominately dilative

Fig. 4. Proposed updated SBTn chart based on Q_{tn} - F_r (solid lines show soil behaviour type boundaries, and dashed lines show boundaries suggested by Robertson 1990).



at large shear strains. Robertson (2010) indicated that the flow liquefaction case histories showed that the suggested boundary (represented by $CD = 70$) was slightly conservative (i.e., soils with some data slightly lower Q_{tn} values could be dilative at larger strains). The dashed line in Fig. 4 shows an approximate lower limit, based on the case histories presented by Robertson (2010) for ideal soils that are predominately dilative at large strains that can be represented by the following simplified expression:

$$(11) \quad CD(\text{lower bound}) = 60 = (Q_{tn} - 9.5)(1 + 0.06F_r)^{17}$$

In general, it is recommended to apply the upper boundary ($CD = 70$) for most geotechnical interpretation, since this is often slightly conservative. The $CD = 70$ boundary shown in Fig. 4 applies only to ideal soils with little or no microstructure, since some aged and (or) cemented soils can be contractive at large strains but produce relatively high values of Q_{tn} due to the increased stiffness and strength from aging and (or) cementation. Examples will be presented later to illustrate this point.

The boundary between the original (Robertson 1990) SBTn zones 4 (silt mixtures) and 5 (sand mixtures) is the approximate boundary between soils that are either more claylike or more sandlike and can be approximated by $I_c = 2.6$ (Fig. 2). However, the simple circular shape of I_c is not always a good fit to the original boundary, except for predominately young uncemented, essentially normally consolidated (ideal) soils, as suggested by Robertson and Wride (1998).

Schneider et al. (2012) had suggested a more hyperbolic shape (in terms of $\log Q_t$ and $\log F_r$) to better capture the SBT boundaries. Figure 4 shows suggested modified main SBTn boundaries based on a more hyperbolic shape using a modified soil behaviour type index, I_B , defined as

$$(12) \quad I_B = 100(Q_{tn} + 10)/(Q_{tn}F_r + 70)$$

The boundary shown in Fig. 4 represented by $I_B = 32$ represents the lower boundary for most sandlike ideal soils and is similar to the original boundary between SBTn zones 4 and 5 for normally con-

solidated soils. The boundary represented by $I_B = 22$ represents the upper boundary for most claylike ideal soils, and is similar to the original boundary between SBTn zones 3 and 4 for normally consolidated 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 behaviour somewhere between that of either sandlike or claylike 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 2012) and are sometimes referred to as “intermediate soils”. The modified boundaries shown in Fig. 4 are similar to the original boundaries for zones 3–5 in the central part of the chart, where most young uncemented, normally consolidated ideal soils plot.

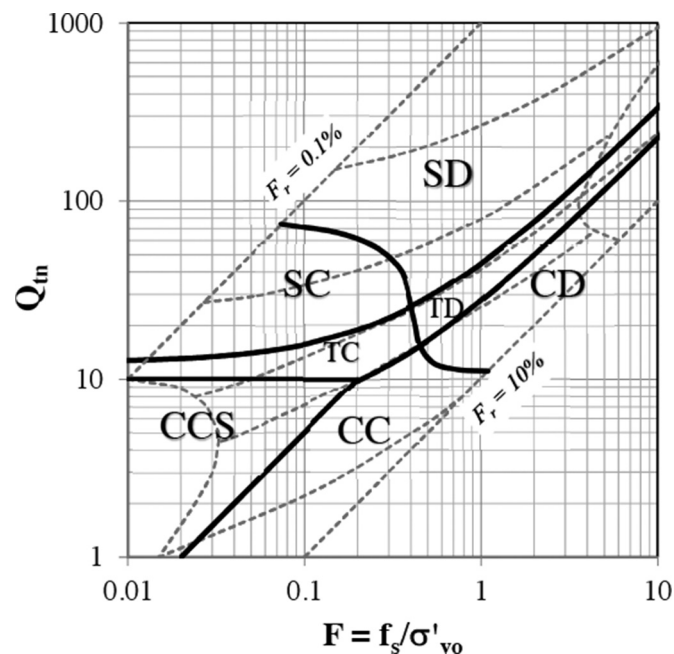
The soil close to the friction sleeve of the cone has experienced very large shear strains and tends to be fully destructured and (or) remolded. Based on this observation, Robertson (2009) had suggested that sensitivity (S_t) in most fine-grained ideal soils could be estimated using the simplified expression:

$$(13) \quad S_t = 7.1/F_r$$

Hence, soils with $F_r < 2\%$ tend to have a sensitivity $S_t > 3$ –4. This boundary has been included in Fig. 4 as an approximate separation between claylike–contractive (CC) soils with moderate to low sensitivity ($S_t < 3$) from claylike–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 Fig. 4, the boundary between CC and CCS is slightly more conservative than the previous boundary suggested by Robertson (1990).

Eslami and Fellenius (1997) had suggested that charts based on Q_t and F_r were not mathematically correct for any statistical analyses, since both Q_{tn} and F_r use q_t . Figure 5 shows the same bound-

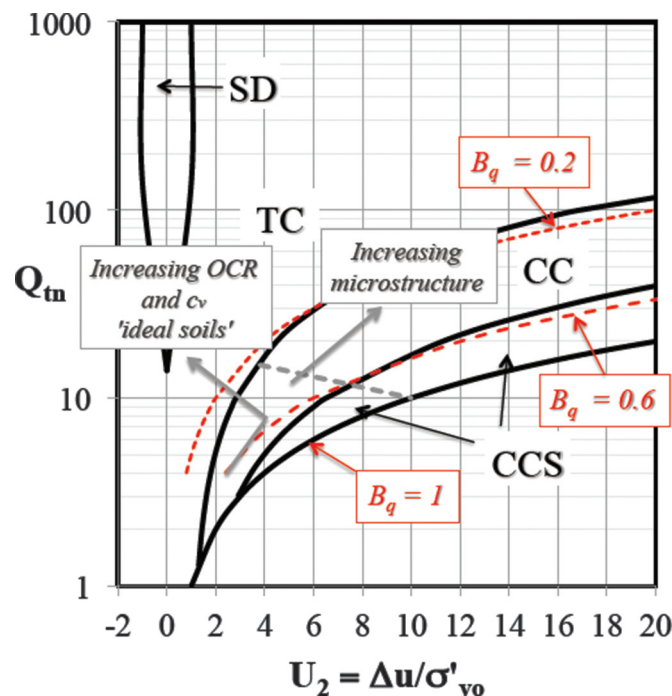
Fig. 5. Proposed alternate format SBTn chart based on Q_{tn} - F (solid lines show proposed new soil behaviour type boundaries, and dashed lines show boundaries suggested by Robertson 1990).



aries as Fig. 4 but represented on a modified SBTn chart based on Q_{tn} and F (where F is the normalized sleeve resistance, $F = f_s/\sigma'_{vo}$). Also shown in Fig. 5 (dashed lines) are the original boundaries suggested by Robertson (1990), for comparison. The format shown in Fig. 5 has the advantage that the normalized parameters are independent, but the disadvantage that data becomes more compressed. For most applications, there is no difference between Figs. 4 and 5 and either format can be used, although the Q_{tn} - F_r format (Fig. 4) is preferred, and for the examples shown later, this format is used. Figure 5 is similar to the earliest normalized CPT "characterization" chart that was suggested by Olsen (1984).

The Q_{tn} - 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 ($Q_t < 10$). Although the basic Q_{tn} - 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 behaviour type. Since Schneider et al. (2008) showed that a slightly better form for the normalized pore pressure is U_2 , Fig. 6 shows a proposed modified Schneider et al. (2012) chart using the same behaviour type terms applied in Fig. 4 but using the generalized normalized cone resistance, Q_{tn} , instead of Q_t . The equations to define the boundaries shown in Fig. 6 were provided by Schneider et al. (2008) but replacing Q_t with Q_{tn} . For most claylike soils, there is little difference between Q_{tn} and Q_t , 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 behaviour in shear at large strains; hence, U_2 tends to reflect the behaviour of the soil in shear at large strain (i.e., destructured). In general, positive U_2 values tend to reflect large-strain contractive behaviour, and negative U_2 values tend to reflect large-strain dilative behaviour. Hence, mostly "contractive" behaviour descriptions are shown in Fig. 6 for positive values of U_2 . Based on the Q_{tn} - F_r chart (Fig. 4), 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. 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 behaviour at large strains. Hence, if

Fig. 6. Proposed updated Schneider et al. (2008) chart based on Q_{tn} - U_2 with proposed new soil behaviour type boundaries (B_q lines in red). [Colour online.]



data plot in the region represented by $Q_{tn} > \sim 12$ 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 Fig. 6. This will be illustrated later with some examples. 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 behaviour. For a given soil, increasing OCR can be associated with increasing c_v ; hence, ideal soils with increasing OCR tend to show a decrease in U_2 combined with an increase in Q_{tn} , as illustrated in Fig. 6.

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 Fig. 6. 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 Fig. 6. 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. 4) and Q_{tn} - U_2 (Fig. 6) 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 three charts (Figs. 3, 4, 6) should be used together to improve soil classification. Figure 3 (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), Fig. 4 (Q_{tn} - F_r) should apply. Figure 6 (Q_{tn} - U_2) can be used primarily in fine-grained soils, when V_s data are either not available or as a supple-

ment to Fig. 4, to evaluate if soils have significant microstructure and to evaluate large-strain behaviour, provided reliable pore-pressure measurements are made.

The following section will illustrate the main features of the updated approach using mostly published SCPTu data from well-documented sites. Schneider and Moss (2011) have essentially demonstrated the effectiveness of the $Q_{tn}-G_o/q_t$ chart for a wide range of coarse-grained soils, and Schneider et al. (2008) have essentially demonstrated the effectiveness of the $Q_{tn}-U_2$ chart for a wide range of fine-grained soils; hence, the focus will tend to be more on the $Q_{tn}-F_r$ chart, although data will be presented on all three charts for each example site.

Case history examples

Selected representative case histories are presented to identify specific features that illustrate key points. Table 1 presents a summary of the selected deposits used to evaluate the proposed modified SBTn charts. The sites shown in Table 1 were selected to represent a wide range of deposits ranging from very recent fill and (or) tailings to older very stiff soil or soft rock. The deposits have been listed by increasing value of K_G^* that essentially capture increasing microstructure. The SCPTu data will be presented in the three normalized charts ($Q_{tn}-F_r$, $Q_{tn}-U_2$, and $Q_{tn}-I_G$).

Figures 7–9 present a summary of the average values for all the case history example deposits on the $Q_{tn}-I_G$, $Q_{tn}-F_r$, and $Q_{tn}-U_2$ charts, respectively. Figure 7 ($Q_{tn}-I_G$) shows that the value of $K_G^* = 330$ suggested by Schneider and Moss (2011) provides a good separation between soils with little or no microstructure and soils with significant microstructure. Also, $K_G^* = 330$ appears to separate predominately silica-based soils from carbonate-based soil. Figure 8 ($Q_{tn}-F_r$) shows that, in general, soils with significant microstructure tend to plot in the dilative region of the chart. Figure 9 ($Q_{tn}-U_2$) shows that fine-grained contractive soils with significant microstructure plot in a region defined approximately by $U_2 > 0$ when $Q_{tn} = 20$ and $U_2 > 10$ when $Q_{tn} = 10$.

The following examples were selected to illustrate the variation and trends in the SCPTu data in the selected deposits.

Soils with little or no microstructure

Figure 10 shows SCPTu data to a depth of 40 m at the KIDD site near Vancouver, British Columbia, which was part of the CANLEX research project (Robertson et al. 2000; Wride et al. 2000). The soils are essentially normally consolidated, uncemented Holocene-age natural (silica-based) deposits from the Fraser River delta and represent both sandlike and claylike soils in one profile. The SCPTu was carried out at a part of the site where the soils from a depth of about 4 to 22.7 m are relatively uniform medium dense fine sand overlying uniform sensitive marine clay with the water table at a depth of 1 m, and V_s measurements were made every 0.5 m starting at 3.25 m. The sand (4–22.7 m) plots predominately in the sandlike–dilative (SD) region consistent with the detailed results from the CANLEX project with an average $K_G^* = 214$. The underlying normally consolidated sensitive marine clay (22.9–40 m) plots in the claylike–contractive–sensitive (CCS) region on both the $Q_{tn}-F_r$ and $Q_{tn}-U_2$ charts, with an average $K_G^* = 215$. The K_G^* values are consistent with the Holocene-age, with no evidence of cementation. Figure 10 shows that the original descriptions used by Robertson (1990) also provide a good classification of both the sand and clay deposits. The CPT data transitions from the sand to the clay between 22.7 and 22.9 m and is incorrectly classified. The issue of data in transition from sandlike to claylike was discussed by Robertson (2009).

Figure 11 shows SCPTu data to a depth of 35 m at a site in the San Francisco Bay area near Vallejo. The test location was pre-drilled using a hand auger to a depth of 1.5 m after which the SCPTu was started and V_s measurements were made every 1 m starting at a depth of 2 m. The groundwater table is at a depth of 2 m. The site

is composed of about 1.5 m of fill overlying Young Bay Mud (YBM) to a depth of around 12 m. General details about YBM are provided by Bonaparte and Mitchell (1979). At this location, the YBM is late Holocene-age and lightly overconsolidated below a depth of 4 m ($OCR < 1.5$) with a desiccated surface crust due to groundwater fluctuations. Below the YBM is Old Bay Clay (OBC) to the final depth of the SCPTu at 36 m. The SCPTu data from 2 to 11 m in the YBM are shown in Fig. 11a and from 12 to 35 m in the OBC in Fig. 11b. The average K_G^* value for the YBM at this site is around 85, consistent with the young geologic age and lack of microstructure. The OBC has slightly higher K_G^* values of around 300, consistent with the older geologic age (late Pleistocene). The YBM below the desiccated crust plots in the claylike–contractive–sensitive (CCS) region on the $Q_{tn}-F_r$ chart and in the claylike–contractive (CC) region of the $Q_{tn}-U_2$ chart. The YBM has a sensitivity of around 4–6 based on field vane tests, which is consistent with the classification of CCS. The difference in classification between the $Q_{tn}-F_r$ and $Q_{tn}-U_2$ charts is partly due to a somewhat slow pore-pressure response in the upper 11 m of the profile after recording small negative pore pressures in the desiccated crust. In the YBM desiccated crust, the normalized cone resistance (Q_{tn}) moves higher and plots in the transitional soil region (TC and TD) of the $Q_{tn}-F_r$ chart. As desiccation increases closer to the ground surface, with associated increase in apparent OCR, the CPT data plots higher on both the $Q_{tn}-F_r$ and $Q_{tn}-U_2$ charts. As Schneider et al. (2008) identified the U_2 values decrease with increasing Q_{tn} due to a more dilative behaviour and increasing c_v (and possible partial drainage during the CPT). Close to the ground surface (at a depth of about 2 m), some of the SCPTu data plot in the sandlike–dilative (SD) region due in part to the very dilative behaviour of the very stiff desiccated clay and the almost drained penetration during the CPT; however, most of the SCPTu data for the desiccated crust plot in the transitional–dilative (TD) region. The OBC is more stratified and variable as indicated by the wide range in normalized CPT values shown in Fig. 11b. Much of the OBC data plot on the boundary between claylike and transitional soils and contractive to dilative behaviour, consistent with the variable nature of this predominately stiff overconsolidated sandy clay.

Figure 12 shows SCPTu data between depths of 4 and 20 m at the Bothkennar site in the UK (Hight and Leroueil 2003). The Bothkennar soil is young estuarine clayey silt with an organic content of 3%–8%. It has sensitivity, measured by the fall cone, of between 5 and 13 and an apparent OCR of 1.4–1.6. The clay is described as slightly “structured” due to possible organic cementation (Hight and Leroueil 2003). Based on the SCPTu data, the K_G^* values are around 240, and the data plot mostly in the CC and CCS regions of the $Q_{tn}-F_r$ chart and the CC region in the $Q_{tn}-U_2$ chart. The somewhat higher K_G^* values are consistent with some small amount of microstructure. Although the Bothkennar clay is described as structured, the existing CPT-based empirical correlations provide good estimates of undrained shear strength, OCR, and sensitivity, which are consistent with K_G^* values less than 330 and the observation that the level of microstructure is minor.

Figure 13 shows SCPTu data from the loose silica-based sand site at Holmen in Norway (Lunne et al. 2003) from 3 to 20 m. The Holmen sand is young (2000–3000 years ago) and very loose due to rapid deposition in quiet water in front of the delta formed by the Drammen River. The SCPTu data plot in the sandlike–contractive (SC) region of the $Q_{tn}-F_r$ chart, with essentially no excess pore pressures ($U_2 \sim 0$) and K_G^* values of about 155.

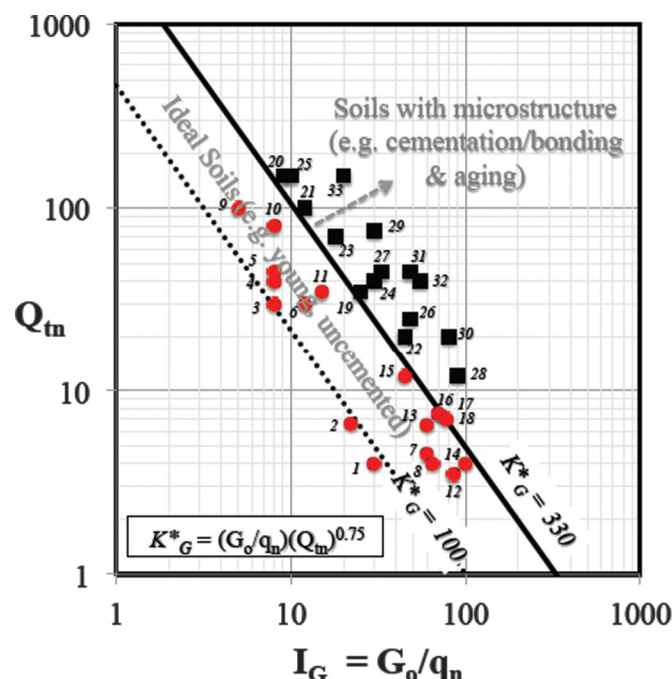
Figure 14 shows SCPTu data from the Madingley site near Cambridge, UK (Lunne et al. 1997) from 2 to 12 m. The Madingley site is underlain by Gault clay that is very stiff overconsolidated fissured clay of the Cretaceous period (~110 million years ago) with $OCR > 10$. The high overconsolidation ratio is derived from significant stress removal with no evidence of cementation. The SCPTu data correctly plot predominately in the claylike–dilative (CD)

Table 1. Selected representative examples.

Reference No.	Site	Description	Geologic age	Q_{tn}	F_r (%)	U_2	I_G	K_G^*	Reference
1	San Francisco (USA)	Young Bay mud, NC-LOC clay	Holocene	4.0 (3.0–5.0)	1.0 (0.5–2.0)	2.0 (1.5–3.0)	30 (25–35)	85 (70–100)	P.K. Robertson (personal files)
2	Burswood Perth (Australia)	NC-LOC clay	Holocene	6.5 (5.5–8.0)	2.0 (1.0–3.0)	2.5 (1.5–3.5)	22 (18–30)	90 (75–122)	Randolph (2004)
3	Syncrude (Canada)	NC tailing sand	Recent	30 (20–65)	0.7 (0.4–1.0)	0	8 (6–10)	105 (100–110)	Wride et al. (2000)
4	Highmont (Canada)	NC tailing sand	Recent	40 (20–70)	0.3 (0.1–0.6)	0	8 (6–12)	130 (120–140)	Wride et al. (2000)
5	LL Dam (Canada)	NC tailing sand	Recent	45 (30–60)	0.4 (0.1–0.8)	0	8 (6–12)	140 (130–150)	Wride et al. (2000)
6	Holmen (Norway)	Drammen River, NC sand	Holocene	30 (20–60)	0.4 (0.2–0.7)	0	12 (10–18)	155 (125–205)	Lunne et al. (2003)
7	Onsoy (Norway)	NC-LOC clay	Holocene	4.5 (4.0–5.0)	1.8 (1.2–2.2)	3.0 (2.5–3.5)	60 (50–70)	185 (155–215)	Lunne et al. (1997)
8	Tabarao (Brazil)	NC-LOC clay	Holocene	4.0 (3.0–5.0)	1.3 (1.1–1.5)	1.7 (1.0–2.2)	65 (50–80)	185 (140–225)	Schnaid and Odebrecht (2015)
9	University of British Columbia, McDonald Farm (Canada)	Fraser River, NC sand	Holocene	100 (50–150)	0.3 (0.2–0.5)	0	5 (3–9)	200 (100–300)	Campanella et al. (1983)
10	KIDD (Canada)	Fraser River, NC sand	Holocene	80 (40–150)	0.6 (0.3–1.0)	0.1 (–0.1–0.5)	8 (5–12)	214 (130–310)	Wride et al. (2000)
11	J-Pit (Canada)	NC tailing sand	Recent	35 (20–50)	0.7 (0.6–1.0)	0	15 (9–25)	215 (170–270)	Wride et al. (2000)
12	KIDD (Canada)	Fraser River, NC-LOC clay	Holocene	3.5 (3.0–4.0)	1.1 (0.9–1.4)	3.0 (2.0–3.5)	85 (60–110)	215 (155–280)	Wride et al. (2000)
13	Bothkennar (UK)	LOC silty clay	Holocene	6.5 (5.5–8.0)	1.6 (1.0–2.5)	3.6 (2.5–4.5)	60 (50–70)	240 (205–290)	Hight and Leroueil (2003)
14	University of British Columbia, McDonald Farm (Canada)	Fraser River, NC-LOC clay	Holocene	4.0 (3.0–5.0)	1.8 (1.5–2.2)	2.5 (2.0–3.0)	100 (80–120)	280 (230–320)	Campanella et al. (1983)
15	San Francisco (USA)	Old Bay clay, OC clay	Late Pleistocene	12.0 (5–30)	2.5 (1.5–5.0)	2.5 (–1.5–5.0)	45 (30–100)	300 (195–600)	P.K. Robertson (personal files)
16	Venice (Italy)	NC-LOC clayey silt	Holocene	7.5 (5.0–10.0)	2.0 (1.3–2.8)	1.0 (0.1–1.8)	70 (60–80)	315 (270–350)	Simonini (2004)
17	Amherst (USA)	Varved clay	Late Pleistocene	7.5 (6.5–9.0)	1.5 (0.8–3.0)	5.0 (3.0–6.0)	70 (50–90)	325 (225–400)	DeGroot and Lunenegger (2003)
18	Ford Center (USA)	LOC clay	Pleistocene	7 (5.5–9.0)	3.0 (2.5–43.5)	3.8 (3.0–4.5)	78 (70–90)	330 (300–380)	Finno et al. (2000)
19	Madingley (UK)	HOC clay	Cretaceous	35 (30–45)	4.5 (3.0–6.0)	–1	25 (15–30)	360 (330–430)	Lunne et al. (1997)
20	Griffin (USA)	Dense sand	Pleistocene	150 (80–400)	0.6 (0.3–1.0)	0	9 (4–20)	380 (215–800)	Safner et al. (2011)
21	Tangiers (Morocco)	Calcareous sand	Recent	100 (60–200)	0.2 (0.1–0.8)	0	12 (7–20)	380 (215–560)	Debats et al. (2015)
22	Campinas (Brazil)	Residual soil	—	20 (15–25)	6.0 (5.0–8.0)	1 (0–4)	45 (40–60)	425 (375–565)	De Mio et al. (2010)
23	Dubai (United Arab Emirates)	Calcareous sand	Recent	70 (60–80)	0.2 (0.1–0.3)	0	18 (15–21)	435 (360–500)	P.K. Robertson (personal files)
24	Porto (Portugal)	Residual soil	—	40 (30–50)	5.0 (3.0–7.0)	0	30 (20–40)	475 (320–640)	De Mio et al. (2010)
25	Ledge Point (Western Australia)	Calcareous sand	Holocene	150 (80–300)	0.5 (0.2–0.8)	0	10 (5–20)	500 (330–800)	Schneider and Lehane (2010)
26	Houston (USA)	Beaumont, OC clay	Pleistocene	25 (20–30)	4 (3.5–4.5)	–0.5 (–1.0–0)	48 (35–50)	535 (425–640)	Mayne (2009)
27	Atlanta (USA)	Piedmont, residual soil	—	45 (25–65)	2.2 (1.2–3.0)	–0.5 (–1.0–0)	33 (25–40)	570 (470–640)	Mayne (2009)
28	Charleston (USA)	Cooper Marl, calcareous clay	Oligocene	12 (8–25)	0.6 (0.4–1.4)	6 (4.5–11)	90 (50–200)	580 (330–1200)	Camp et al. (2002)
29	Los Angeles (USA)	Fernando, siltstone	Miocene	75 (60–100)	1.5 (1.0–2.0)	33 (20–40)	30 (17–40)	635 (510–1020)	P.K. Robertson (personal files)
30	Anchorage (USA)	Bootlegger Cove, HOC clay	Pleistocene	20 (10–50)	1.3 (1.0–2.0)	1.7 (1.0–5.5)	80 (50–110)	750 (450–1400)	Mayne and Pearce (2005)
31	Windsor (USA)	Calcareous cemented clay	Miocene	45 (40–50)	2.5 (2.0–3.0)	12 (5–16)	48 (40–65)	830 (690–1150)	Ku et al. (2013)
32	Calgary (Canada)	Glacial clay till	Pleistocene	40 (35–50)	3.0 (2.5–3.5)	5 (0–7.0)	55 (40–70)	850 (600–1100)	Mayne and Woeller (2008)
33	Newport Beach (USA)	Monterey, sandstone-siltstone	Miocene	150 (50–400)	2.5 (0.8–5.0)	1 (0–15)	20 (8–200)	860 (350–4000)	Bastani et al. (2014)

Note: Values shown are mean values, with ranges in parentheses. NC, normally consolidated; LOC, lightly overconsolidated ($OCR < 4$); OC, overconsolidated; HOC, heavily overconsolidated ($OCR > 4$).

Fig. 7. Proposed Q_{tn} - I_G chart with case history examples (details in Table 1). (Red circles are young uncemented silica-based soils; black squares are soils with microstructure or calcareous.) [Colour online.]



region of the Q_{tn} - F_r chart but plot close to the sand region of the Q_{tn} - U_2 chart, due to the negative values of U_2 . This trend of small or negative U_2 values was illustrated in Fig. 6 for ideal soils with high OCR. The average $K_G^* = 360$ indicates some microstructure consistent with the significant age of the deposit but no cementation. The K_G^* value of 360 puts this clay close to the boundary between a soil with little or no microstructure and a soil with significant microstructure. Given that the existing empirical correlations provide a reasonably good estimation of soil behaviour (Lunne et al. 1997), the clay can be considered to be close to the limit of the suggested boundary for ideal soils.

The previous examples, where $K_G^* < 330$, were soils with little or no microstructure and where the proposed SBTn charts provided general good classification in terms of behaviour type.

Soils with significant microstructure

The following examples illustrate soils with significant microstructure, where $K_G^* > 330$ and where the CPTu data does not always fit well on the SBTn charts.

Figure 15 shows SCPTu data to a depth of 50 m at the Cooper River Bridge site in Charleston, South Carolina (Camp et al. 2002), where the soil below a depth of 22 m is Cooper Marl, which is a stiff calcareous plastic clay or silt of Eocene to Oligocene age (~30–40 million years ago). The SCPTu data from 22 to 50 m plot predominately in the transitional-contractive (TC) region of the Q_{tn} - F_r chart and the claylike-contractive (CC) region of the Q_{tn} - U_2 chart with an average $K_G^* = 580$. Dissipation tests provided t_{50} values ranging from 90 to 850 s that indicate predominately undrained cone penetration. Data from a nearby site in the same Cooper Marl but at a shallow depth (less than 9 m) have higher values of $Q_{tn} > 40$ and plot more in the SC and SD regions and with values of U_2 as high as 50. The high K_G^* and U_2 values indicate significant microstructure consistent with cementation from the high carbonate content (Camp et al. 2002). The high Q_{tn} values suggest a relatively high apparent OCR (>4) and possible dilative behaviour, whereas the high U_2 values show a more contractive behaviour at high shear strains consistent with a cemented soil. The high level of cementation is consistent with a very stiff behav-

our at small strains followed by a contractive behaviour at high shear strains when the cementation is broken and the soil becomes destructured.

Figure 16 shows SCPTu data from a site in Atlanta, Georgia, in the Piedmont residual soil (Mayne 2009). The site served as a test area for instrumented piles (Harris and Mayne 1994) and consists of silty fine sand derived from the weathering of the underlying gneiss and schist bedrock. The SCPTu data from 4 to 18 m plot predominately on the boundary between sandlike-dilative (SD) and transitional-dilative (TD) regions of the Q_{tn} - F_r chart with an average $K_G^* = 520$. The excess pore pressures are generally negative with values close to -100 kPa and hence close to the saturation limit of the sensor. Dissipation tests provided t_{50} values ranging from 20 to 100 s that indicate a potentially partially drained cone penetration. The negative pore pressures and the potential for partially drained penetration can explain why the data plot toward the sand region of the Q_{tn} - U_2 chart. The high K_G^* values indicate significant microstructure consistent with the remaining cementation in the residual soil. Residual soils derived from granite bedrock in Porto, Portugal, have similar high K_G^* values and plot in TD region of the Q_{tn} - F_r chart, whereas data from a more fine-grained clayey silt residual soil in Campinas, Brazil, plot in the CD region of the Q_{tn} - F_r chart (De Mio et al. 2010). The significant microstructure present in many residual soils tends to explain why the current SBTn charts often misinterpret their classification.

Figure 17 shows SCPTu data from a site near downtown Los Angeles composed of a uniform siltstone from 3 to 10 m. The siltstone is part of the Fernando Formation of Pliocene age (~3–5 million years ago). The SCPTu data plot in the sandlike-dilative region (SD) on the Q_{tn} - F_r chart but in the claylike-contractive (CC) region of the Q_{tn} - U_2 chart (note that the U_2 values are between 18 and 40 and are mostly off the scale of the Q_{tn} - U_2 chart at the scale shown). The average $K_G^* = 635$ is consistent with the old age of the deposit combined with the cemented nature of this soft rock. The Q_{tn} - U_2 chart is correctly classifying the behaviour as contractive at large strain that is different to the dilative behaviour suggested by the Q_{tn} - F_r chart. The difference is explained by the significant microstructure (cementation) identified by the high K_G^* values.

Figure 18 shows SCPT data from a site in Tangiers, Morocco (Debats et al. 2015), composed of hydraulically placed calcareous sand. The sand has a high carbonate content ranging from 75% to 95% and is composed of shell fragments and carbonate algae mixed with grains of silica and mica (Debats et al. 2015). The highly compressible nature of the carbonate (shell) mineralogy produces small friction ratio values ($F_r < 0.3\%$) but high small-strain stiffness with $K_G^* \sim 380$. Soils with high carbonate content have a tendency to develop rapid calcium cementation, resulting in some microstructure and high values of K_G^* . However, it is also possible that an apparent microstructure is caused by the highly compressible nature of the carbonate grains. The resulting "microstructure" often makes CPT interpretation difficult using traditional empirical correlations based on predominately silica-based soils.

In some parts of the world where calcareous sands exist, it has become common practice to correct the CPT cone resistance to "equivalent silica-sand" values using a shell correction factor (SCF) where

$$(14) \quad q_{c(ss)} = (SCF)q_c$$

where $q_{c(ss)}$ is the equivalent silica-sand cone resistance.

The SCF has been estimated based on calibration chamber studies that have been costly and provide limited results. Debats et al. (2015) suggested that the SCF could be estimated from SCPT data by adjusting q_c , using an iterative approach with variable SCF values, until the measured V_s agrees with an estimated V_s using

Fig. 8. Proposed updated SBTn chart based on Q_{tn} - F_r with case history examples (details in Table 1). (Red circles are young uncemented soils; black squares are soils with microstructure or calcareous.) [Colour online.]

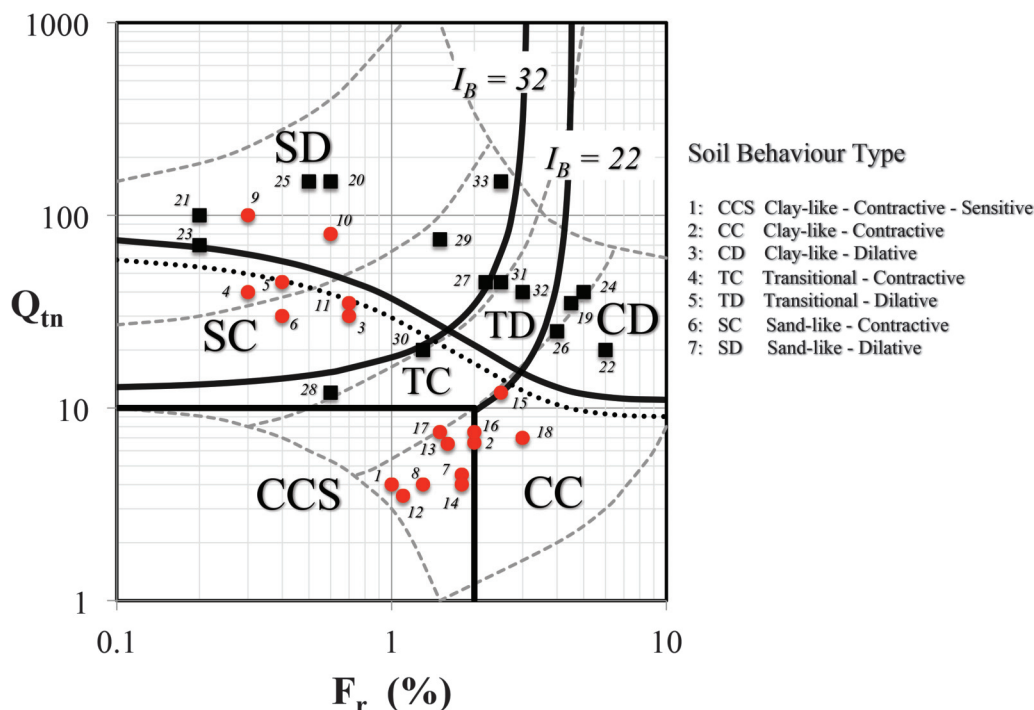
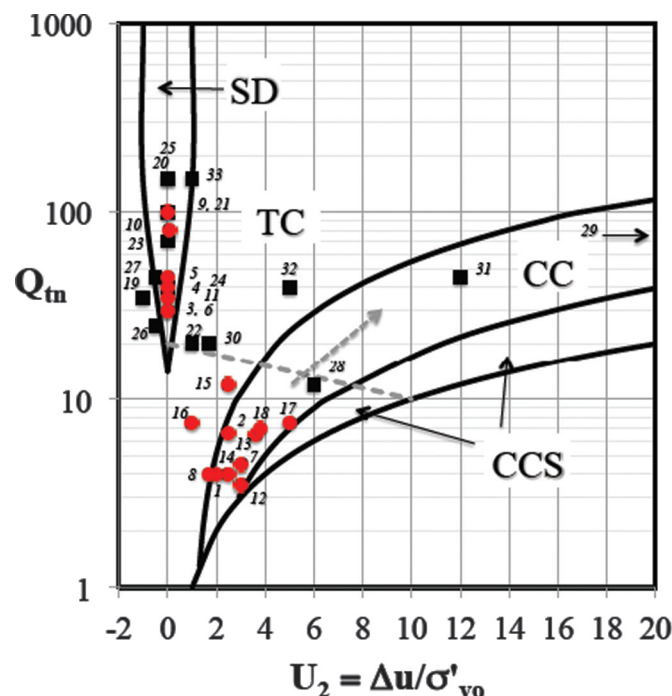


Fig. 9. Proposed updated Schneider et al. (2008) chart based on Q_{tn} - U_2 with case history examples (details in Table 1). (Red circles are young uncemented soils; black squares are soils with microstructure or calcareous.) [Colour online.]



the empirical CPT-based correlation suggested by Robertson (2009) using the adjusted q_c . Since the average K_G^* for most young, uncemented silica-based sands is about 215 (Schneider and Moss 2011), it is also possible to estimate the SCF based on SCPT data using the following simplified relationship:

$$(15) \quad SCF = (K_G^*/215)^{1.334}$$

where K_G^* is the average measured normalized small-strain rigidity index.

Application of eq. (15) is conceptually similar to the approach used by Debats et al. (2015) but avoids the need for iteration to obtain the SCF. For the data shown in Fig. 18, the SCF based on eq. (15) is 2.1 compared to the average value of about 2.0 suggested by Debats et al. (2015).

Discussion

The examples in the preceding section illustrate that the modified Q_{tn} - F_r and Q_{tn} - U_2 charts provide good classification in terms of soil behaviour type for soils with little or no microstructure when $K_G^* < 330$. The Q_{tn} - U_2 chart can be somewhat sensitive to minor loss of saturation of the pore-pressure sensor in some soils, especially in stiffer onshore soils. In soils and soft rock where there is significant microstructure (i.e., structured soils) with $K_G^* > 330$, the classification of soil behaviour type becomes less reliable and some judgment is required. If structured soils and (or) soft rocks are sufficiently fine-grained to develop reliable excess pore pressures, the Q_{tn} - U_2 chart provides a better classification of soil behaviour type at large strains and can be used to identify significant microstructure. Geomaterials with significant microstructure tend to be cemented or bonded and can be very stiff at small strains (producing high Q_{tn} values) but can be contractive at large shear strains (producing high U_2 values) when the cementation is destroyed and the material becomes destructured.

As discussed by Robertson (1990), the addition of dissipation tests to measure the rate of dissipation of any excess pore pressures (e.g., t_{50}) can also aid in the correct classification of soil behaviour, as well as drainage conditions during the CPT. DeJong and Randolph (2012) suggested that when $t_{50} > 50$ s, the cone penetration (1000 mm² cone at standard rate of 20 mm/s) is essentially undrained.

Fig. 10. Example of normalized SCPTu data from 4 to 40 m at the KIDD site in the Fraser River delta near Vancouver, British Columbia. (Orange dots for sand from 4 to 22.7 m; red dots for clay from 22.9 to 40 m; green dots for transition zone from 22.7 to 22.9 m.) $Du_2/\sigma'_v = \Delta u_2/\sigma'_{v0}$. [Colour online.]

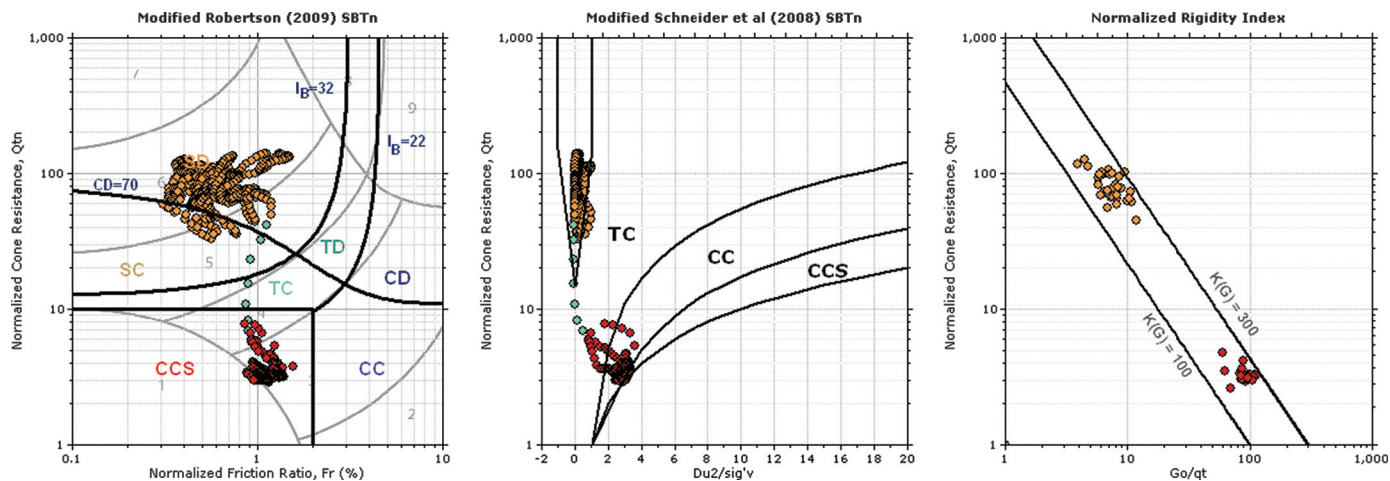


Fig. 11. Example of normalized SCPTu data from San Francisco Bay, California, USA: (a) Young Bay Mud with desiccated crust (2–11 m); (b) Old Bay Clay (12–35 m). [Colour online.]

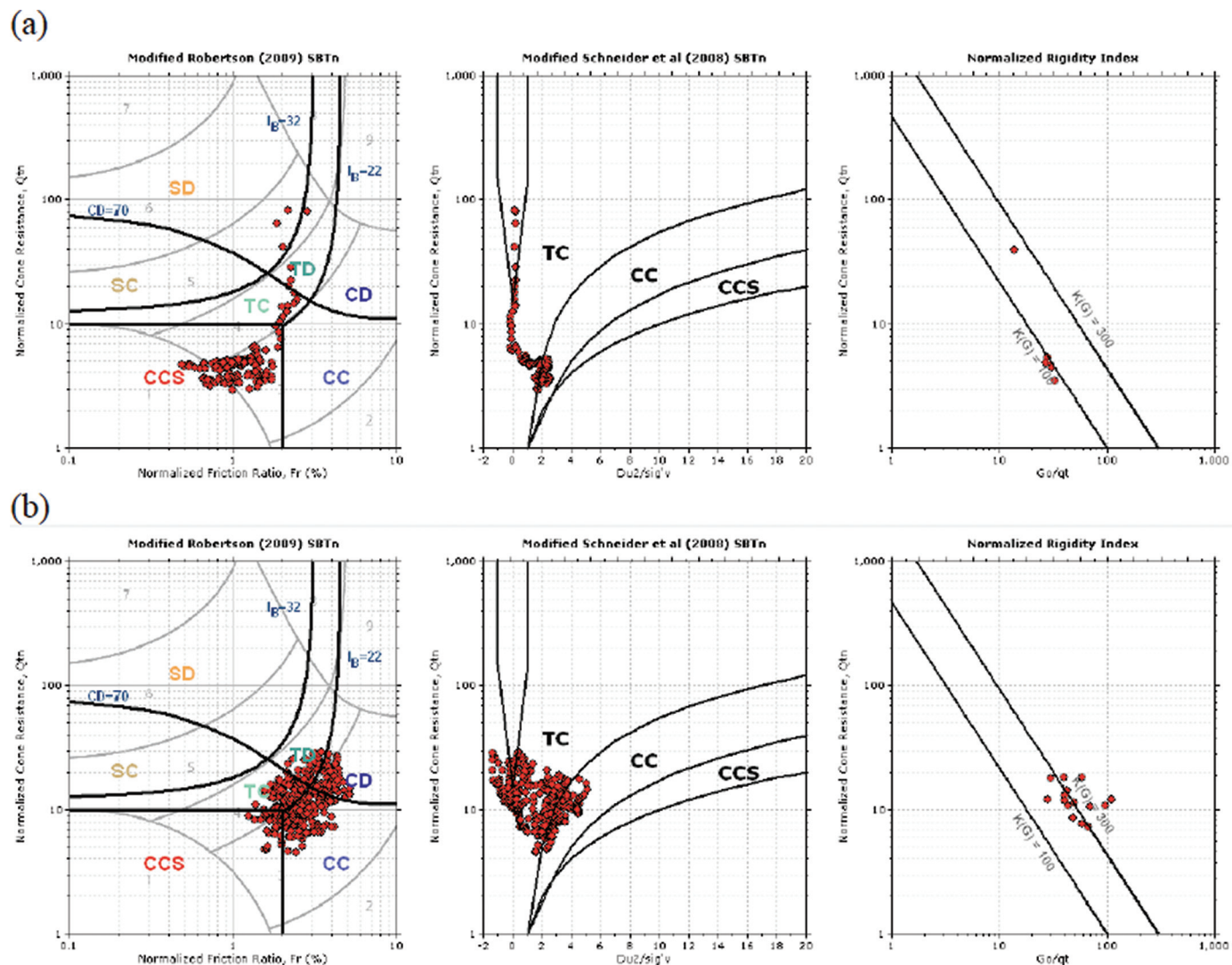


Fig. 12. Example of normalized SCPTu data from 4 to 20 m at the Bothkennar clay site, UK. [Colour online.]

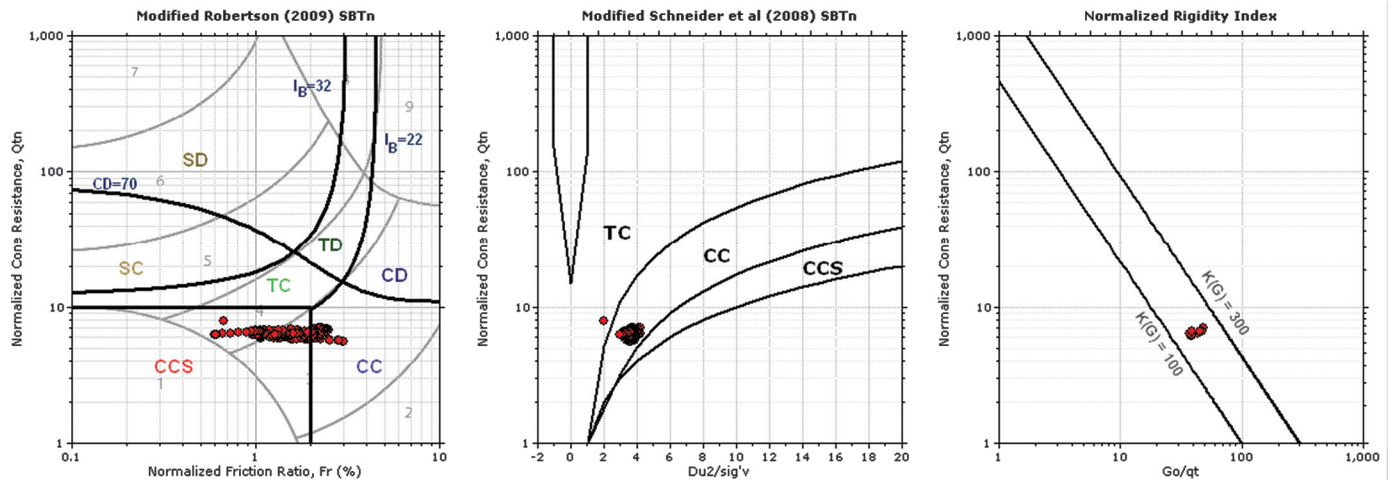


Fig. 13. Example of normalized SCPTu data from 3 to 20 m at the Holmen loose sand site, Norway. [Colour online.]

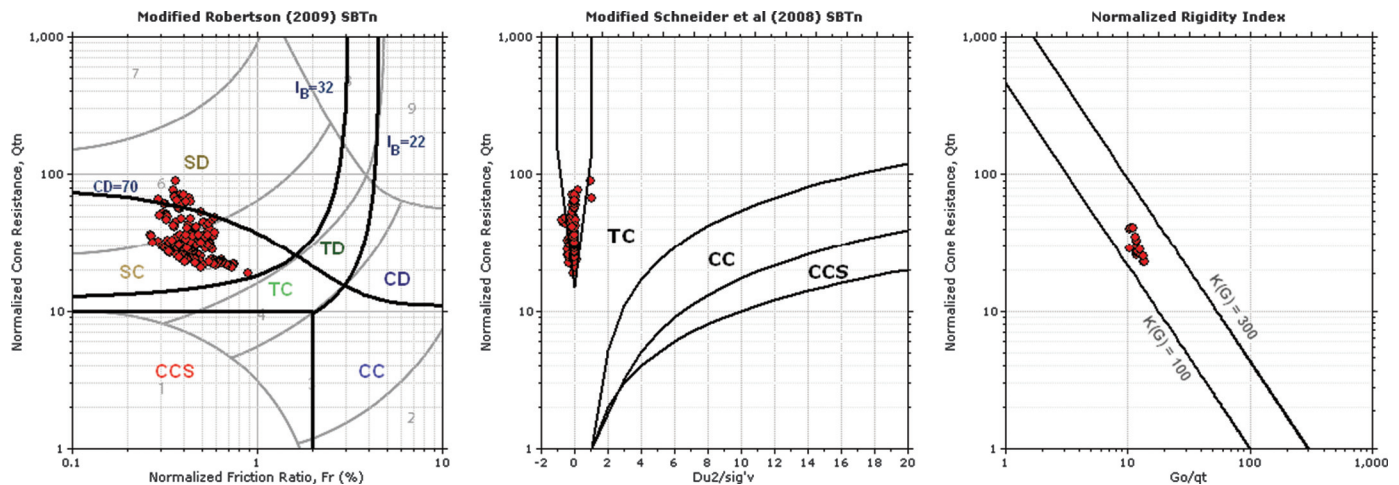
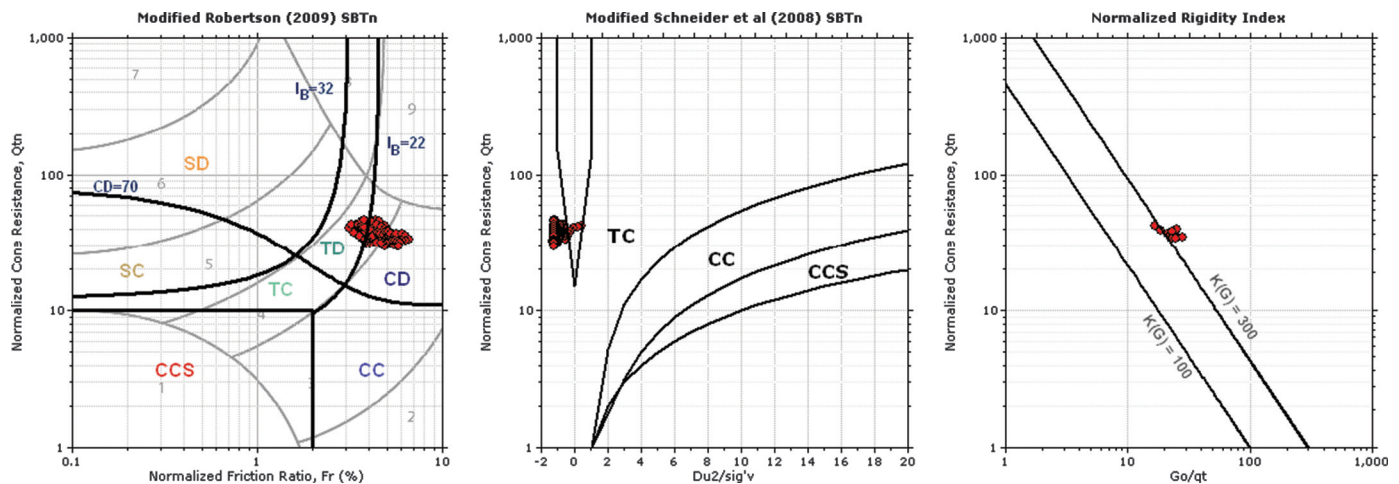


Fig. 14. Example of normalized SCPTu data from 2 to 12 m at Madingley stiff clay site, Cambridge, UK. [Colour online.]



Soil behaviour is controlled by the in situ effective stress; hence, CPT parameters normalized in terms of effective stress can better capture in situ behaviour for classification. However, the normalization process is not always perfect due to uncertainty in estimating in situ stress. Conceptually, any normalization should also account for the important influence of horizontal effective stresses, since pen-

etration resistance is strongly influenced by the horizontal effective stresses. However, this continues to have little practical benefit for most projects without a prior knowledge of in situ horizontal stresses (Robertson 2009). Uncertainty in estimating in situ vertical stresses can also result from uncertainty in soil unit weight (γ) as well as the in situ piezometric profile (u_0). This un-

Fig. 15. Example of normalized SCPTu data from 22 to 50 m in Cooper Marl (calcareous cemented clay), Charleston, South Carolina, USA. [Colour online.]

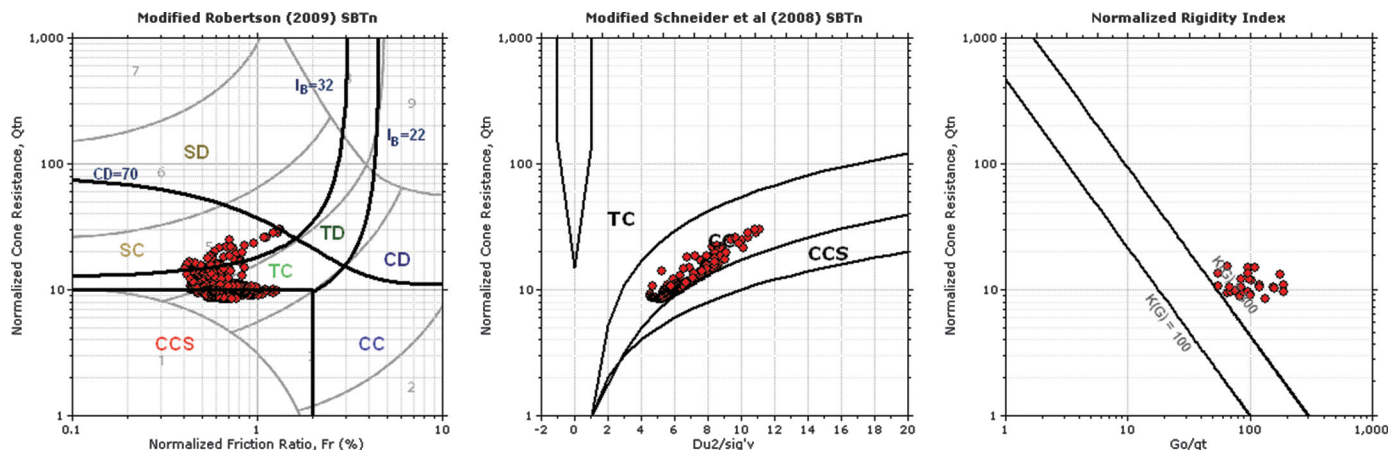


Fig. 16. Example of normalized SCPTu data from 4 to 18 m in Piedmont residual soil at site in Atlanta, Georgia. [Colour online.]

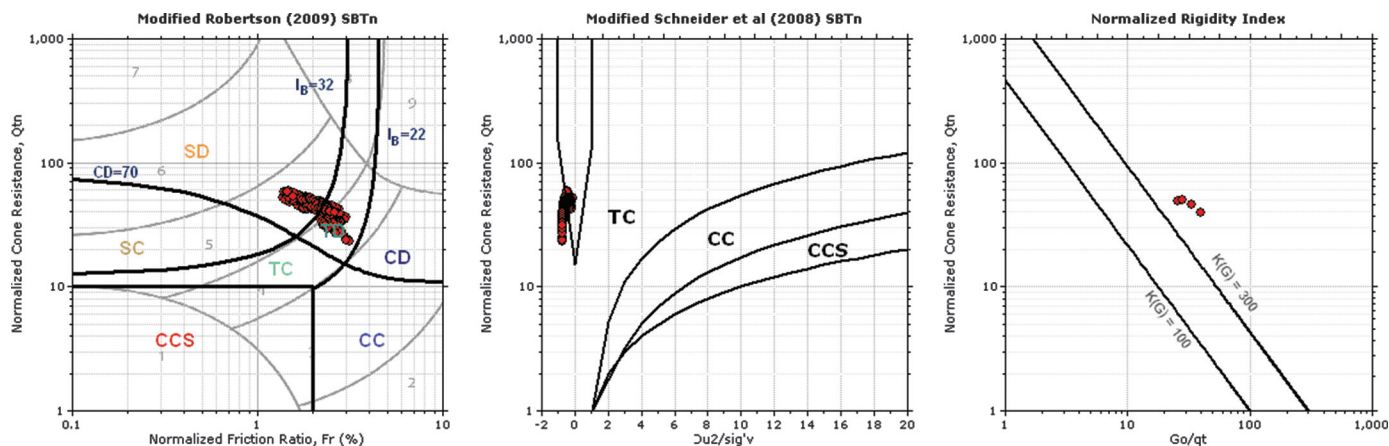
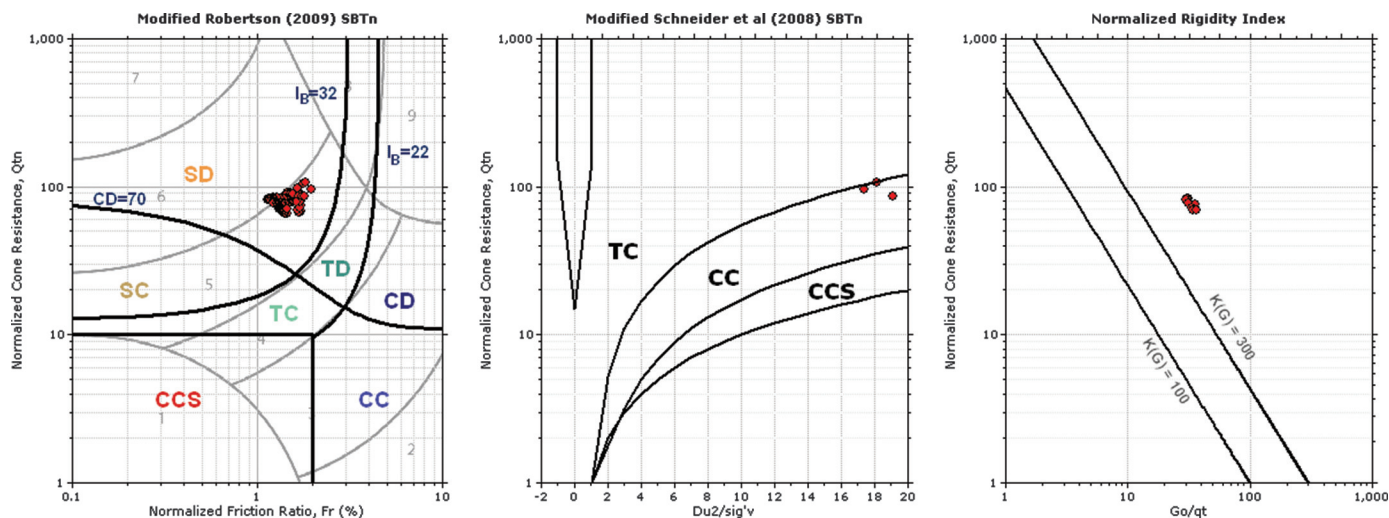


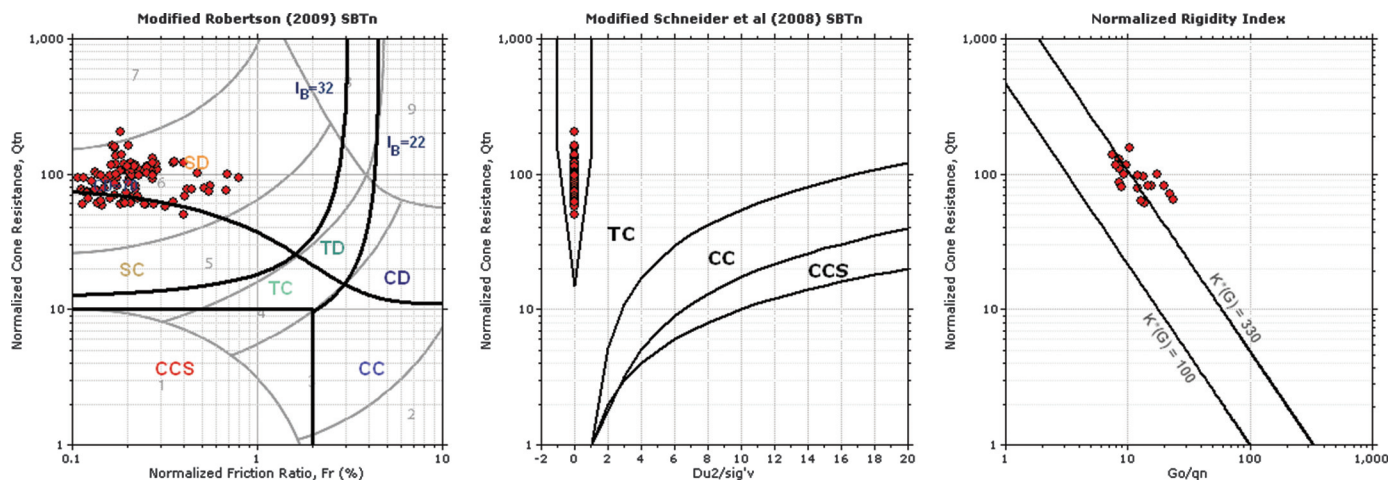
Fig. 17. Example of normalized SCPTu data from 3 to 10 m in siltstone at downtown Los Angeles site, California. [Colour online.]



certainty typically has a minor effect in coarse-grained soils (e.g., SD and SC) where the measured CPT parameters (q_c and f_s) tend to be large relative to σ_{vo} and u_0 . However, the normalized parameters (Q_{tn} , F_r , and I_G) can become sensitive to uncertainty in σ_{vo} and u_0 in very soft fine-grained soils (e.g., CC and CCS). Likewise, the normalized CPT parameters can also be sensitive to measurement uncertainty in soft fine-grained soil where the measured CPT pa-

rameters (q_c and f_s) can be close to the limits of accuracy or repeatability for the CPT equipment (Robertson 2009). One method to identify when the normalized parameters (e.g., Q_{tn} and F_r) may not be correct is when the data plot outside some of the limits shown on the charts. For example, $Q_{tn} < 1$ when pushing in soil is rare and is often the result of an overly high estimate for σ_{vo} . Soft soils with high organic content can have very low soil unit weight

Fig. 18. Example of normalized SCPT data from 4 to 14 m in hydraulically placed calcareous sand in Tangier, Morocco (data from Debats et al. 2015). [Colour online.]



that can result in apparent low values of q_n and Q_{tn} due to overestimated values of σ_{vo} . In soft fine-grained soils, it is also rare to obtain $B_q > 1.2$, since this requires an excess pore pressure higher than the net cone resistance (q_n). Hence, if data plot lower than the line identified by $B_q = 1.0$ in the $Q_{tn}-U_2$ chart, there may be some uncertainty in the low values of Q_{tn} . In soft soils, these uncertainties in estimating σ_{vo} and u_0 often outweigh the uncertainty in horizontal stress ratio (K_0), since variations in K_0 tend to be captured by the empirical correlations (e.g., OCR). The normalized parameter I_C requires in situ shear wave velocity measurements that can be difficult to obtain accurately in the upper 1–2 m and at depths greater than about 50 m using a down-hole method like the SCPT. Likewise, scatter in I_C can also result from scale effects between the measured V_s and CPT measurements in heterogeneous deposits.

The CPT measurements taken over water often require special care to ensure correct data normalization. Typically, CPT work over water is referenced to the mudline (i.e., point where soil starts), since the effective stress is zero at the mudline and the cone zero load readings are also taken at the mudline. Hence, σ_{vo} and u_0 are also referenced to mudline (i.e., equivalent to assuming piezometric surface at the mudline). If the CPT zero load readings are taken at the water surface (e.g., for shallow over water work), the cone (q_t and u_2) will measure the water pressure when lowered through water (with $f_s \sim 0$). Hence, σ_{vo} and u_0 must be selected to produce $\sigma'_{vo} = 0$ at the mudline, which may require using the unit weight of water for the section of CPT data in water.

Calculation of Q_{tn} requires an iterative procedure to determine the stress exponent (n) using I_C . The proposed modified SBTn chart based on Q_{tn} and F_r contains a modified soil behaviour type index (I_B) to define the main boundaries between sandlike and claylike behaviour; however, it is recommended to continue using I_C in the iterative procedure to determine Q_{tn} , since I_C adequately captures the variation of soil behaviour to estimate the stress exponent as well as other current applications of I_C (e.g., Robertson 2009 and Mayne 2014).

Summary and conclusions

Updated and modified charts have been proposed to estimate soil behaviour type based on either CPT, CPTu, or SCPTu data. Ideally, these charts should be used in conjunction with the traditional textural-based classification system (e.g., USCS) based on samples. The charts utilize normalized parameters in an effort to capture in situ soil behaviour. New behaviour descriptions are suggested in an effort to be consistent with the concept of a behaviour type classification. The method is based on CPT (i.e., electric cones of either 500,

1000, or 1500 mm² area) at the standard penetration rate of 20 mm/s. The $Q_{tn}-F_r$ chart does not apply to data from a mechanical CPT, since both the tip and sleeve resistance values from a mechanical CPT can be different than an electric CPT.

Since soil behaviour can be complex, it is recommended to apply multiple CPT-based measurements to improve behaviour classification. The SCPTu offers the opportunity to obtain anywhere from three to six independent measurements to improve classification. However, classification is still possible using either two (q_c and f_s) or three (q_c , f_s , and u_2) measurements from the standard CPT or CPTu. Classification based on only two measurements (q_c and f_s) is generally less reliable and should be limited to predominately silica-based, young, uncemented soil (i.e., ideal soil). Classification is improved if three measurements are used (q_c , f_s , and u_2) especially in more fine-grained soils. Ideally, classification should be based on four measurements (q_c , f_s , u_2 , and V_s), since this allows identification of possible microstructure. In fine-grained soils, dissipation tests that provided an added measurement (t_{50}) are also valuable and recommended where possible. Dissipation tests in coarse-grained layers, where 100% dissipation can be rapid and cost effective, are recommended so that the correct equilibrium piezometric pressure (u_0) can be determined.

The geologic history of the deposit is always a helpful starting point for correct classification. If the geologic history supports the potential that the soils are predominately silica-based, young (i.e., Holocene to Pleistocene age) and likely uncemented, the basic SBTn charts based on Q_{tn} and F_r will generally provide reliable classification.

The link between behaviour characteristics (e.g., strength, stiffness, and compressibility) that is reflected in the CPT measurements and physical characteristics (e.g., grain size and plasticity) can be very good in soils with little or no microstructure (i.e., ideal soils). Hence, it can be important to identify the level of microstructure in a deposit. If V_s (and hence G_0) data are available, ideally using SCPT, the proposed $Q_{tn}-I_C$ chart can be used to evaluate the level of microstructure in a deposit with supporting evidence from the geologic background. The modified $Q_{tn}-U_2$ can also be helpful in fine-grained soils where CPT penetration is essentially undrained.

If soils have little or no microstructure, the proposed SBTn charts and existing empirical correlations (e.g., Robertson 2009) tend to apply and can provide reasonable estimates of soil behaviour. However, if soils have significant microstructure (e.g., $K'_G > 330$), the proposed SBTn charts and most existing empirical correlations may not always apply and site or geologic specific modifications may be required.

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