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# GIG 9.0 Guidance Notes on Cone Penetration Testing

## Part 1: Cone Equipment, Operation and Soil Behaviour Type Interpretation

### 1. INTRODUCTION

A Cone Penetration Testing (CPT) is carried out by pushing a calibrated cone vertically into the ground and measuring the forces applied on its conical tip, the friction on the sleeve of the cone and, if using a piezo-cone, the pore water pressure is measured during course of the cone penetration.

### 2. HISTORY

Probing with a cone attached to rods for the evaluation of the stratum and strength of the ground has been practised since about 1917. In 1932, the CPT as recognised today was developed in the Netherlands with an outer casing through which an inner rod could freely move. At the base of the rod, a 10 $cm^2$ cone with a 60° apex angle was attached and the outer pipe and inner rod could be pushed down.

### 3. EVOLUTION

In 1948, the basic mechanical cone was developed (Fig 1) and this cone is still in use today as the standard cone in some areas of the world.

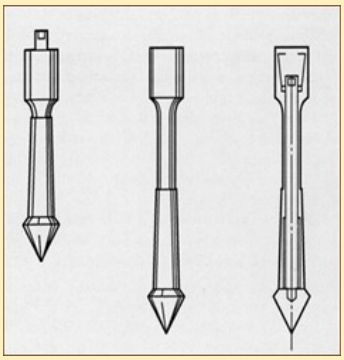

Fig. 1. End Cone Mantle.

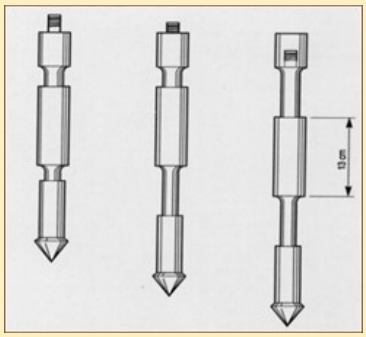

Fig. 2. Cone and Friction Sleeve. (Begemann, 1953)

The test still required an inner rod and outer casing with the outer casing attached to the cone and the inner rod used to push the cone ahead of the casing to measure the soil resistance.

In 1953, Begemann patented the first cone to measure the local skin friction (Fig 2). Begemann (1965) also proposed that the friction ratio could be used to classify soil layers in terms of soil type.

In 1965, Fugro in conjunction with the Dutch State Research Institute developed the electric cone with the shape and dimensions of the cone forming the basis for the international reference test procedures that are still valid today. In 1974, an additional development provided the measurement of pore water pressure (Fig 3).

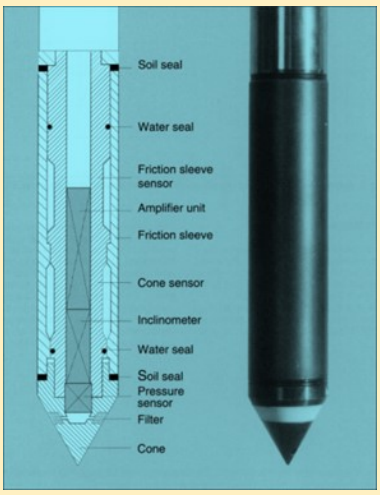


Fig. 3. The Electrical Piezocone

Edited and Rewritten by Ir. Raymond S. M. Chan (Version 2020)

The location of the pore pressure filter has been developed in several positions. In practice, the filter position (Fig 4) is normally located in the cone face ($u_1$) and close behind the cone ($u_2$) for marine and land projects. The triple element piezocone is equipped with a third pore pressure filter ($u_3$) that is above the friction sleeve. However, this cone is normally used for research purpose. The diameters of the cones of 35.7mm (I.e., 10cm$^2$ in area) and 43.7mm (15cm$^2$) are commonly used in the market.

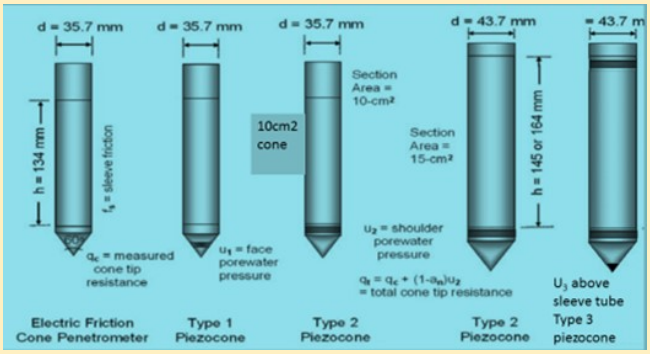


Fig. 4. Typical dimensions and types of piezocones

It is important that the verticality of the cone is known whilst the test is being carried out so that the true vertical depth can be calculated, and the data corrected.

The basic cone measures tip resistance and sleeve friction with the addition of a pore pressure sensor if detailed parameter calculations are required.

## 4. SPECIAL FUNCTIONS OF CONES

In addition to the basic cone measurements many additional parameters are now routinely measured with specially designed cones, including:

- Seismic Cone - Dynamic Properties by measurement of P and S wave velocities to obtain small strain shear modulus.
- Pressuremeter Cone - Stiffness measurement by measurement of probe expansion versus pressure to obtain shear strength and modulus.
- Electrical Conductivity - Electrical conductivity of the ground for pipeline corrosion assessment and detection of saline ground water.
- Magnetometer Cone - Measurement in changes of the ground magnetic field for detection of UXO or base of sheet pile walls.
- T Bar and Ball Cones – Mainly used offshore for profiling strength of very soft clays and silts for pipeline design.
- Temperature Cone - Measurement of Insitu ground temperature often used for measurement of landfill temperatures.
- Thermal Conductivity Cone - Insitu measurement of the ground temperature conductivity for design of buried power cables.
- Vibration Cone – Measurement of vibration from piling activities to monitor sensitive underground structures.
- Natural Gamma Cone - Measurement of Natural Radiation within the ground, especially useful for detecting Chalk interface.
- Laser Induced Fluorescence Cone - Use of optical laser light for screening of hydrocarbon derived contamination of soil and water.
- Membrane Interface Cone - Uses a range of sensors to detect contamination by organic compounds and chlorination hydrocarbons in the soil.
- XRF Cone - Use of X – Ray fluorescence to detect metal contaminates within the ground.
- HPT Cone - Hydraulic profiling tool for the continuous logging of relative hydraulic conductivity.

## 5. CPT EQUIPEMENT

In order to carry out a CPT you need equipment that can provide the force and reaction required to push the cone into the ground at the constant rate of 2 cm per second. In the 1940's, this required hand-installed ground anchors and manual cranking to wind the cone into the ground with the readings taken manually from dial gauges (Fig 5). Today the tests are carried from a wide variety of equipment ranging from six-wheel drive trucks to crawlers between 1 tonne and 20 tonnes for land use using hydraulic rams with data captured on computers Fig 6 to 7).

In the nearshore environments, 20 tonne hydraulic rams are used on jackup platforms or the seabed units, Roson, from floating vessels (Fig 8 to 11).

With land units, every effort is made to ensure that each test is carried out vertically, with many units fitted with hydraulic levelling jacks. On marine units, while a jackup will level the unit legs, the Manta, or similar units, sits on the seabed and has no self-levelling capability. It is therefore very important that a continuous measurement of cone verticality is recorded during a test.

Fig. 5. A CPT Test Carried Out in 1940's

Fig. 6A. A Modern 20 Tonne Wheel Drive CPT Truck

Fig. 6B. Offloading a 20 Tonne Crawler from Carrier

Fig. 7 A One Tonne CPT Crawler Using its Own

Fig. 8. Seabed Manta Unit Being Deployed on a Moon-pool Barge

Fig. 9. CPT Ram Set Working on a Jackup Barge

Fig. 10. CPT Units Coming in a Variety of Sizes

Fig. 11 Typical Vessel Used for CPT

## 6. Auxiliary Equipment To CPT

When the CPT unit is lowered to sit on seabed, it is commonly experienced that it takes long time to wait for the CPT unit to be stable and stationery with angle of inclination from vertical to be less than 5 degrees. The seabed intervention requires a great deal of skill in un-predictable conditions. In order achieve the acceptable inclination without long time operation, an interchange-able levelling frame (Fig 12A) is simply mounted on the bottom of the CPT unit, and it can be put on and taken off easily. The frame has the following main characteristics:

- handles slope up-to 20 degrees.
- no loss off reaction weight.
- levels automatically.
- locks during test.
- inclination control.

Fig 12A An Interchangeable Levelling Frame Mounted to the Seabed CPT unit

There is another design of levelling for the seabed CPT unit. It is the installation of three hydraulic legs equipped with circular steel plates at the bottom of the seabed CPT unit, and it is shown in Figure 12B.

Fig. 12B Three Hydraulic Legs with Circular Steel Plates are Equipped at the Bottom of the Seabed CPT unit

## 7. Testing Procedures

The cone end resistance and local sleeve friction are registered by calibrated load cells in the cone. The pore water pressure is recorded by means of a calibrated pressure transducer located behind the piezocone tip. In order to ensure pore water measurements are not affected by the presence of air in the measuring transducer, a de-airing procedure is carried out prior to each test. This comprises injecting silicon oil (or a suitable miscible and viscous fluid) into the measuring parts of the cone and using a pore pressure filter which has been saturated in silicon oil under vacuum. As a final precaution, a rubber membrane is placed over the filter to prevent the risk of de-saturation before the cone enters the soil.

Alternatively, the cone can be de-aired inside a de-air chamber (Figure 13) that filled with silicon oil. During de-airing, air bubbles will be released from the cone and gradually vanished with time. Once the de-airing is completed, the top lid of the de-air chamber can be opened and a rubber membrane should be put on the cone tip and cone body to maintain air tight.

The signals from the measuring devices are transmitted by an umbilical cable through hollow push rods to a computer. During the test, the computer displays instantaneous and continuous graphical records on its screen of cone end resistance, local side friction, and pore water pressure, and these are plotted out after the test. The data are recorded on magnetic media at 2 cm depth intervals and this facility provides for subsequent automatic computer-controlled processing and plotting of cone end resistance, local sleeve friction, friction ratio and pore water pressure. The rate of penetration during all tests is generally kept constant at 2 cm per second but it can be at 1cm per second as Geomil's CPT system does. Readings of side friction, tip resistance, and pore water pressure shall be obtained at depth increments not exceeding 20mm, and preferably not under 10mm.

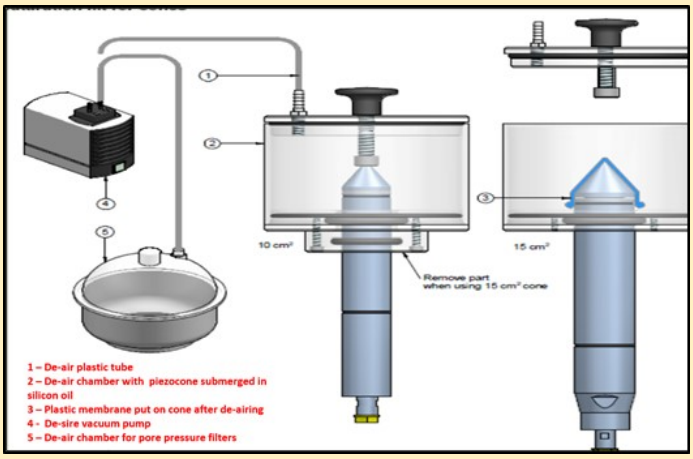


Fig. 13 De-air Equipment for cone and cone filters by vacuum pump device

CPT Test shall terminate when satisfying criteria as specified in the respective contract at the depth below ground or seabed level, or at a lesser depth if any of the following criteria are satisfied:

(1) reaching the maximum capacity of the thrust machine, reaction equipment and/or measurement sensors. It is generally the maximum capacity of 20 tonnes being adopted,
(2) cone tip resistance equal to or exceeding 50MPa or specified.
(3) cone inclination exceeds 15º degrees from vertical.
(4) a sudden increase in penetrometer inclination at 1 degree per metre.
(5) mechanical failure of the test rods likes excessive buckling during penetration.

It is occasionally found that a few of the contracts specified that 5m of completely weathered granite (I.e., CDG) shall be reached. It is in fact that the specification of this kind is generally impracticable, and it is not recommended. It is because that a few of the above-mentioned items should have been reached before the termination in the CDG layer has been reached.

## 8. Specifications and Standards

- ISO Standard, ISO 22476, Part 1 (2012)
- ISSMFE International Reference Test Procedures for CPT, 1989 (Base on new ISO Standard) CEN TC341- WG2 Standard for CPT and CPTU (CEN/ISO 22476-1)
- Swedish Geotechnical Society Recommendation Standard for CPT (1993)
- Norwegian Geotechnical Society Guidelines, 1995
- American Standard, ASTM D5778 – 12 (2012)
- BS: 1377 Part 9,1990 (No longer used, and generally replaced by BS EN 22476-1 2000)
- ISO 19901-8: 2014.

In essence, recommendations regarding the geometry of the cone are that a 60° cone with a face area of 10 $cm^2$ should be used. Also, enlarged cones with a base area of 15 $cm^2$ are utilised. The cone is pushed into the ground at a constant rate of penetration of 2 cm/s with an allowable deviation (e.g., ±5 mm/s).

## 9. Applications of CPT

The results of a CPT test are used to assess the soil types and their distribution and can provide initial assessments of soil strength and, when combined with additional cone modules, can provide information on dynamic properties, stiffness and detailed stratigraphy.

This information can be used to plan an optimum borehole programme with a selective sampling regime, determining estimated shear strength of clays, estimate settlement and bearing pressures in granular soils, pile capacity and pile tip levels and dynamic compaction control.

## 10. CPT for Basic Soil Characterisation

### Static cone penetration test interpretation using cone resistance $q_c$ and sleeve friction $f_s$

Extensive research has indicated that the ratio of local sleeve friction to cone end resistance (friction ratio - $R_f$) can provide a guide to the mechanical properties of soils which assists in identifying the soil behaviour type. The results of various research studies collated by Meigh (1987) have been produced in graphical form and a modified version for British soils by Erwig, is presented in Figure 14. This is the basis of the interpretation of estimated soil types from the basic cone and friction cone.

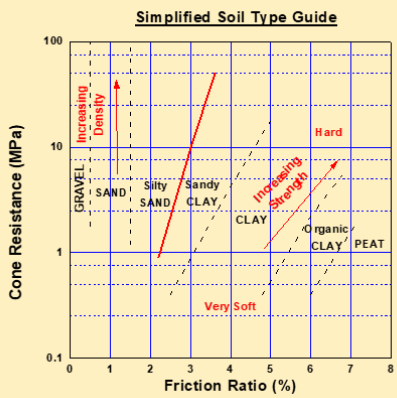


Fig. 14. Estimating Soil Type from Friction Cone

As discussed above, one of the major applications of the CPT is for soil characterisation and profiling. Figures 15 and 16 illustrate typical plots for the test results from a friction cone and piezocone for inferring soil profiling and identification.

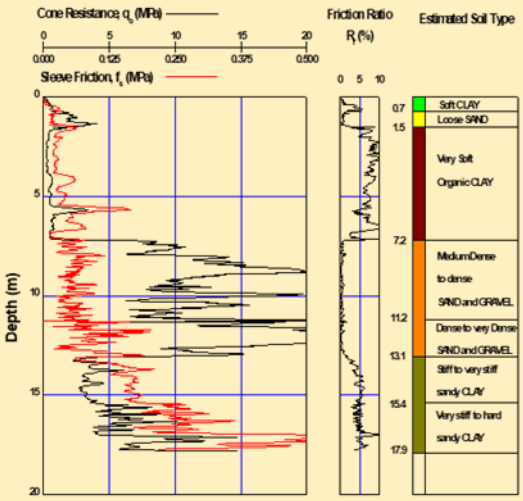


Fig. 15. Report Plot with Soil Type from a Friction Cone

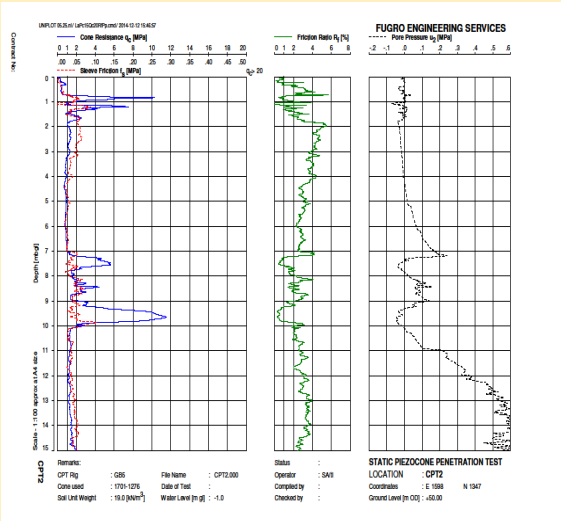


Fig. 16. Report Plot with Pore Pressure Measurement from a Piezocone

## 11. Interpretation of Soil Behaviour Type (SBT) and Normalized Soil Behaviour Type (SBTn)

The Soil Behaviour Type (SBT) charts as shown in Figure 17 provide a guide to mechanical characteristics like strength, stiffness and compressibility of soil. However, most CPT-based SBT classification systems use textural-based descriptions, such as sand and clay that always causes some confusion in geotechnical practice.

SBT charts are different from the Unified Soil Classification System, USCS (ASTM D 2487-11) and the traditional particle size classification soil from Geoguide 3 adopted in Hong Kong. The SBT charts are based on physical characteristics of grain-size distribution and plasticity (Atterberg Limits) measured samples. The charts divide soils into two groups: coarse-grained (sand and gravel), and fine-grained (silt and clay). The coarse-grained soils are those have more than 50% of soil retained on or above the #200 sieve (>0.075mm). The fine-grained soils are those have 50% of soil or more passing the #200 sieve, and they are further grouped based on low or high plasticity. The classification of soil based on the SBT charts have resulted in some confusion in geotechnical practice.

In general, the SBT charts are appropriate for CPT at depth of not greater than 20m. However, the improvement in CPT equipment such that it can penetrate to deeper level of not shallower than 60 to 80m in soil, the SBT chart becomes inappropriate to be used as increase in depth will be in result of increase in overburden pressure and $q_c$ value in an abnormal extent.

Therefore, the Normalized Soil Behaviour Charts (SBTn) as shown in Figure 18 have been developed and suggested by Robertson (1990) that $Q_t$ should be adopted instead of $q_t$. Normalization is often required for very shallow and deep soundings. The CPT-based normalized soil behaviour type (SBTn) method suggested by Robertson (1990) is based on the following normalized parameters:

$Q_t = (qt - \sigma_{vo}) / \sigma_{vo}'$

$F_r = [(fs / (q_t - \sigma_{vo})] 100\%$

$B_q = (u_2 - u_0) / (q_t - \sigma_{vo}) = \Delta u / (q_t - \sigma_{vo})$

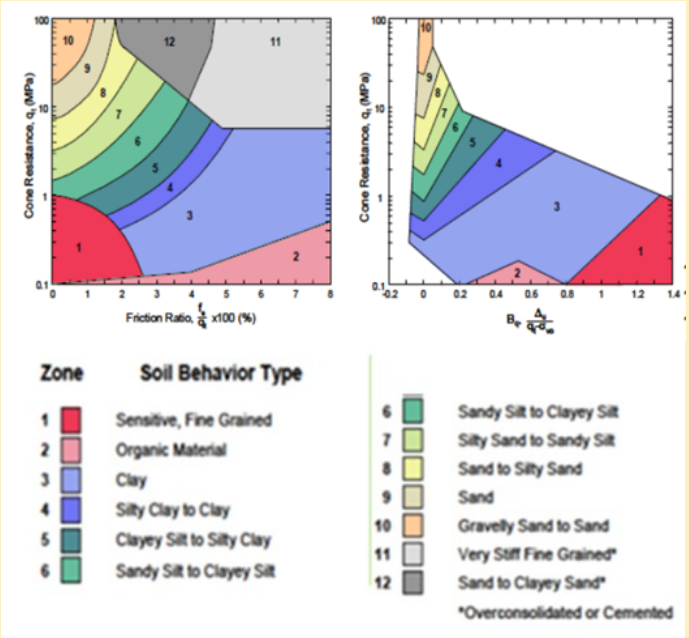


Fig. 17 Robertson (1986) CPT SBT Classification

The SBTn charts suggested by Robertson (1990) are based on either $Q_t$ – $F_r$ and $Q_t$ – $B_q$ but is recommended that the $Q_t$ – $F_r$ chart is generally more reliable especially for onshore geotechnical investigations where the CPT pore pressure results are comparatively less 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.

In general, the normalized charts (SBTn) provide more reliable identification of SBT than the non-normalized charts, although when the in-situ vertical effective stress is between 50 kPa to 150 kPa there is often little difference between normalized and non-normalized SBT.

Since soils are essentially frictional and both strength and stiffness increase with depth, normalized parameters are more consistent with in situ soil behaviour.

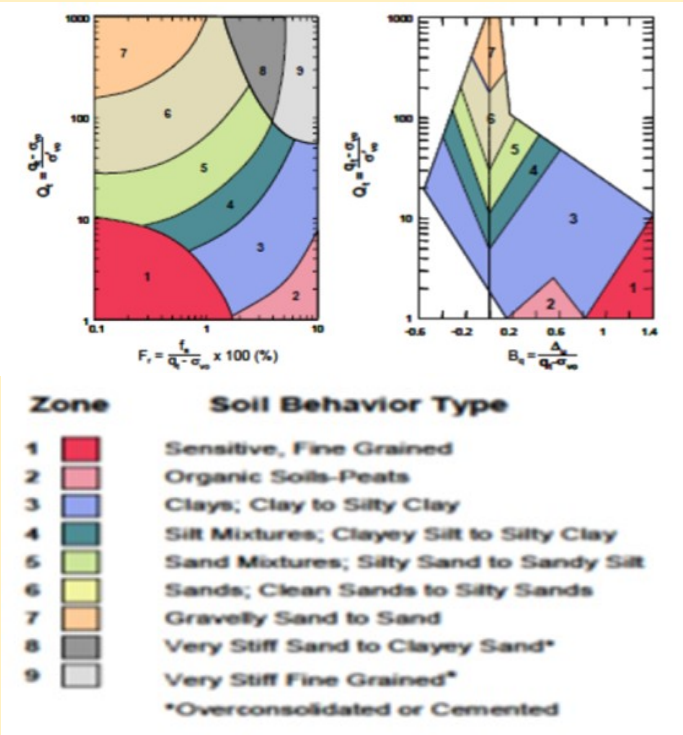


Fig. 18 Robertson (1990) CPT Normalised $SBT_N$ Soil Classification Based on Normalised Cone Resistance and Friction Ratio

The comparison of Soil Behaviour Type Zones between SBT and SBTn is shown in Figure 19.

| 1986 Basic | | Soil Behaviour Type (SBT) | 1990 Normalised | |
|---|---|---|---|---|
| Legend | Zone | | Zone | Legend |
| | 1 | Sensitive, fine grained | 1 | |
| | 2 | Organic soils: peat, clay | 2 | |
| | 3 | CLAY | 3 | |
| | 4 | CLAY – Clay to silty clay | 4 | |
| | 5 | SILT mixtures - Clayey silt to silty clay | 4 | |
| | 6 | SILT – Sandy silt to clayey silt | 5 | |
| | 7 | Fine SAND mixtures – Silty sand to sandy silt | 5 | |
| | 8 | SAND – Sand to silty sand | 6 | |
| | 9 | SAND – Coarse to medium sand | 7 | |
| | 10 | Gravel mixtures – Gravel to gravelly sand | 7 | |
| | 12 | Very compact sand to clayey sand | 8 | |
| | 11 | Very stiff fine grained/Hard clay silt weak rock | 9 | |

Fig 19. Comparison between SBT and SBTn

## 12. Soil Behaviour Type Index $I_c$

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 types (Refer to Figure 20). It was further modified by Robertson and Wride (1998) the definition of $I_c$ to apply to the Robertson (1990) $Q_t$ – $F_r$ chart, as defined by

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

Robertson *et al* (1990) suggested that soils that have the SBTn index $I_c$ of smaller than 2.5 are generally cohesionless where the cone penetration is generally drained, and soils that have $I_c$ of greater than 2.7 are generally cohesive where the cone penetration is generally undrained. Cone penetration in soils with $I_c$ lie between 2.5 and 2.7 are often partially drained.

Robertson and Wride (1998) had suggested that $I_c$ of 2.6 is an approximate boundary between soils that is either more sand-like or more clay-like. It is based on cyclic liquefaction case histories that are limited to mainly silica-based ideal soils of normal consolidation in essential. However, experience has shown that the $I_c$ = 2.6 boundary is not always effective in soils with significant microstructure (i.e., structured soil).

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.

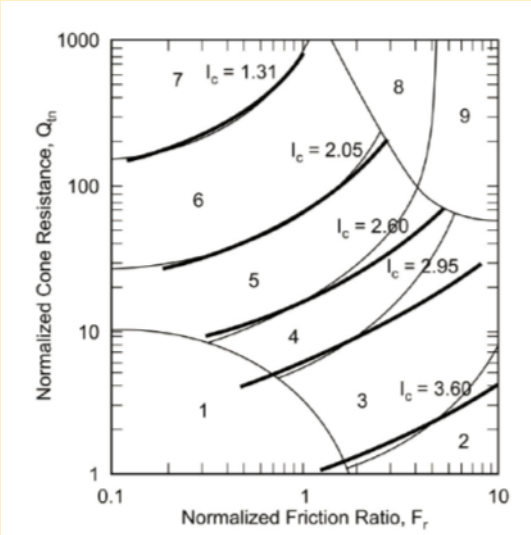


Fig. 20 Soil Behaviour Index Superimposed on SBTn chart

## 13. Updated Normalized Soil Behaviour Type Chart

Since the early normalization charts were being developed and based on the theory for clayey soil, and it was found that was inappropriate to adopted in sandy soil. Robertson and Wride (1998) and updated by Zhang et al. (2002) suggested an updated normalized equation that are based on soil type, soil density and stress level (i.e., Stress exponent n):

$$Q_{tn} = [(q_t - \sigma_{vo})/ Pa] (Pa / \sigma_{vo}')^{n}$$

where:

$Q_{tn}$ = Normalized dimensionless net cone resistance
$(q_t - \sigma_{vo}) / pa$ = dimensionless net cone resistance
$(Pa / \sigma_{vo}')^{n}$ = stress normalization factor
n = stress exponent that varies with SBTn
Pa = atmospheric pressure in same units as $q_t$ and $\sigma_{vo}$.

It is noted that when n is equal to1, $Q_{tn}$ will be equal to $Q_{t1}$ and equal to $Q_t$. The n value should be equal to 1 and 0.5 in clayey and sandy layers respectively. Zhang et al. (2002) suggested that the stress exponent, n, could be estimated using the SBTn Index, $I_c$, and $I_c$ should be defined using $Q_{tn}$. Robertson (2008) recently updated the stress normalization by Zhang et al. (2002) to allow for a variation of the stress exponent with both SBTn $I_c$ and effective overburden stress using:

$$n = 0.381(I_c) + 0.05 (\sigma_{vo} / Pa) - 0.15$$

where $n \leq 1.0$.

Robertson (2008) suggested that the above modification to the stress exponent would capture the correct state response for soils at high stress level and would avoid the need for a further stress level correction ($K_\sigma$) in liquefaction analyses.

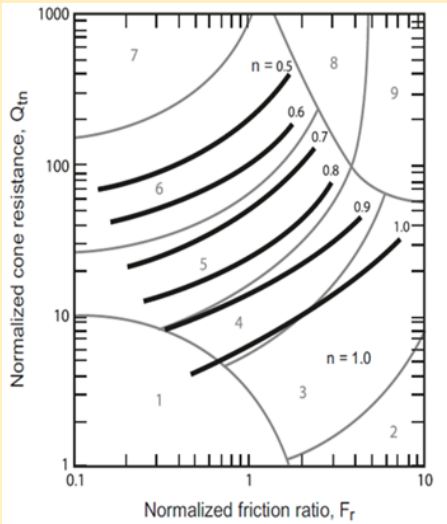


Fig 20 Contours of Stress Exponent n on Normalized SBTn $Q_{tn}$ – $F_r$ Chart

## 14. Abbreviation and Symbols

### Standard Cone Measurements and Factors

| | |
|---|---|
| pwp | Pore water pressure |
| $q_c$ | Measured cone end resistance |
| fs | Sleeve friction |
| α | Net area factor |
| β | value of excessive pwp cone ratio- 0.8 for the face ($u_1$) and 1.0 on the shoulder ($u_2$) |
| $u_o$ | Theoretical hydrostatic pwp relative to ground level acting on cone. |
| $u_1$ | Measured pwp at cone face |
| $u_2$ | Measured pwp at cone shoulder<br>Measured pwp behind friction sleeve |

### Pressures

| | |
|---|---|
| $\sigma_{vo}$ | Total overburden ground pressure |
| $\sigma_{vo}'$ | Effective overburden ground pressure |
| $\sigma_{atm}$ | Atmospheric pressure (Pa) |

**SBT Soil Behaviour Type**

$q_t$ Total cone end resistance corrected for pwp effect where $q_t = q_c + (1-\alpha)\, u_2$

$q_o$ Net cone end resistance where $q_n = q_t - \sigma_{vo}$

$R_f$ Friction ratio of sleeve friction (fs) to measured cone end resistance ($q_c$) or $q_t$

**Normalised Parameters**

SBTn Normalized Soil Behaviour Type

$Q_t$ = Normalized cone resistance
$= ((q_t - \sigma_{vo}) / \sigma_{vo}')$

$F_r$ Normalized Friction Ratio= fs / (qt- σvo)

$B_q$ Pore Pressure Ratio
$= f_s / (q_t - \sigma_{vo})$

**Updated Normalized Parameters**

$Q_{tn}$ Normalized net cone resistance
= ((qt- σvo) / Pa) . (Pa / σvo,)n

n = Stress Exponent factor where
n=1 for clay
n=0.5 for sand

## 15. References

All references should be referred to Part 2 of the Guidance Notes of Cone Penetration Testing, AGS (HK).