Cone Penetration Test (CPT)

A very common method for in situ testing of soils is the Cone Penetration Test (CPT). A cone is hydraulically pushed into the soil at a constant rate, and the resistance to the penetration of the cone, as well as the frictional resistance of a surface sleeve, are continuously recorded.

Today the piezo-cone test (CPTu) is more common, and pore pressure is also measured during penetration. Other variants allow for the measurement of S- and P-wave velocities (seismo-cone), moisture content, soil pH etc.

The Cone Penetration Test provides a continuous profile of the soil stratigraphy, and allows also for continuous evaluation of soil properties; contrary to the SPT test where information about the soil penetration resistance is obtained at specific intervals. However, soil samples are not retrieved for visual inspection and laboratory index testing, as with the SPT test. CPT tests can be performed in very soft clays to dense sands alike, onshore and offshore (Figure 1.).

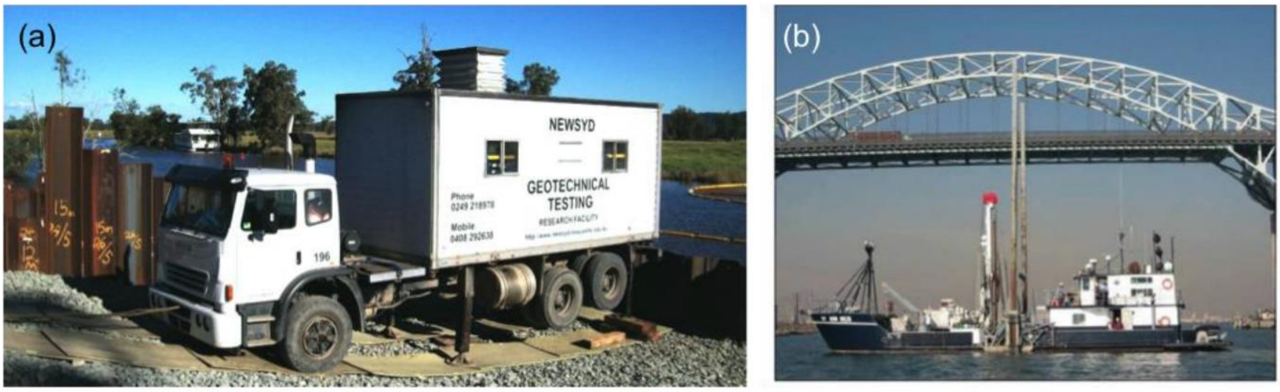


Figure 1: (a) CPT testing equipment fitted on a truck, and (b) Offshore CPT equipment (after Robertson, 2006).

However, CPT is unsuitable for gravelly soils, cemented soils and soft rocks, as the cone cannot penetrate in such formations. During a CPT sounding, the following quantities are measured:

– The cone resistance qc (units: stress), which results from dividing the total force acting on the cone F_{cone} by the projected area of the cone, Ac (Figure 2).

– The sleeve friction fc (units: stress) which results from dividing the total friction force acting on the sleeve F_{sleeve} by the area of the sleeve, As (Figure 2). The diameter of standardized cones is 35.7mm, and their projected area is $Ac=1000mm^2$.

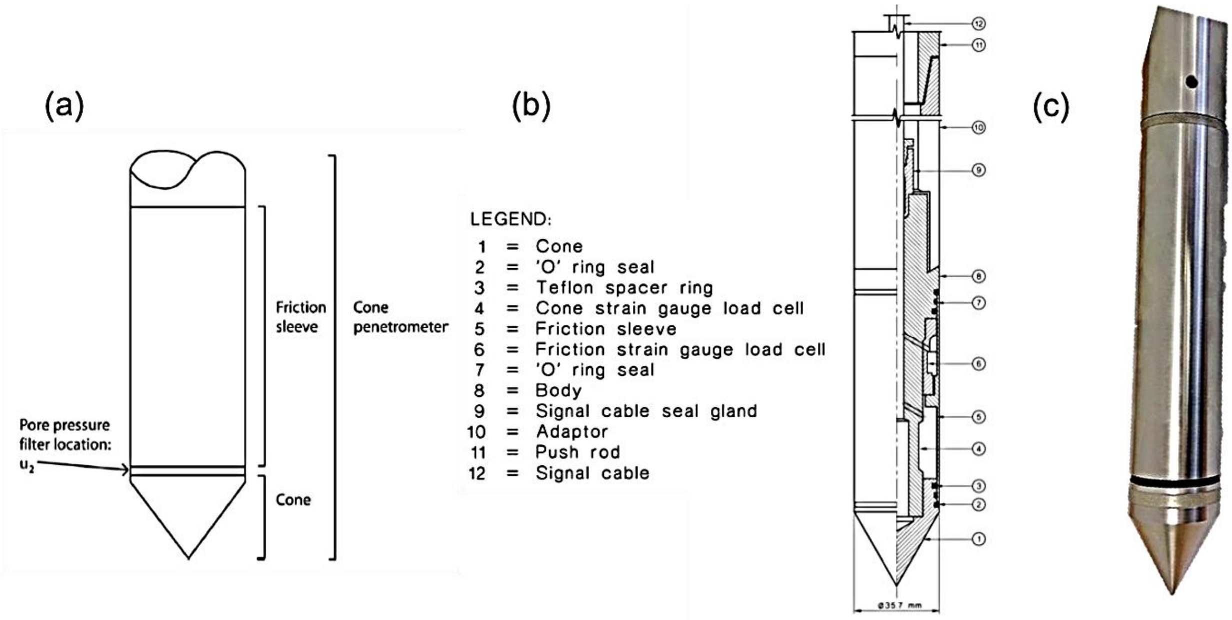


Figure 2: (a) Terminology for cone penetrometers (after Robertson, 2006), (b) Typical electrical friction-cone penetrometer (after AS 1289.6.5.1-1999), and (c) typical cone.

The basic measurements of CPT are:

1. The axial force necessary to drive the cone into the ground (tip resistance, q_c);
2. The axial force generated by adhesion or friction (friction resistance, f_s); and
3. The pore pressure developed as penetration proceeds (u).

The major applications of CPT are:

1. Profiling the soil layers and identification of soil type;
2. Determining pore pressure;
3. Determination of in situ geotechnical soil parameters;
4. Providing direct empirical assessment of foundation performance and soil liquefaction.

Advantages of CPT:

- Fast and continuous profiling
- Repeatable and reliable data (not operator-dependent)
- Economical and productive
- Strong theoretical basis for interpretation

Disadvantage of CPT:

- High capital investment
- Requires skilled operators
- No soil sample
- Penetration can be restricted in gravel/cemented layers

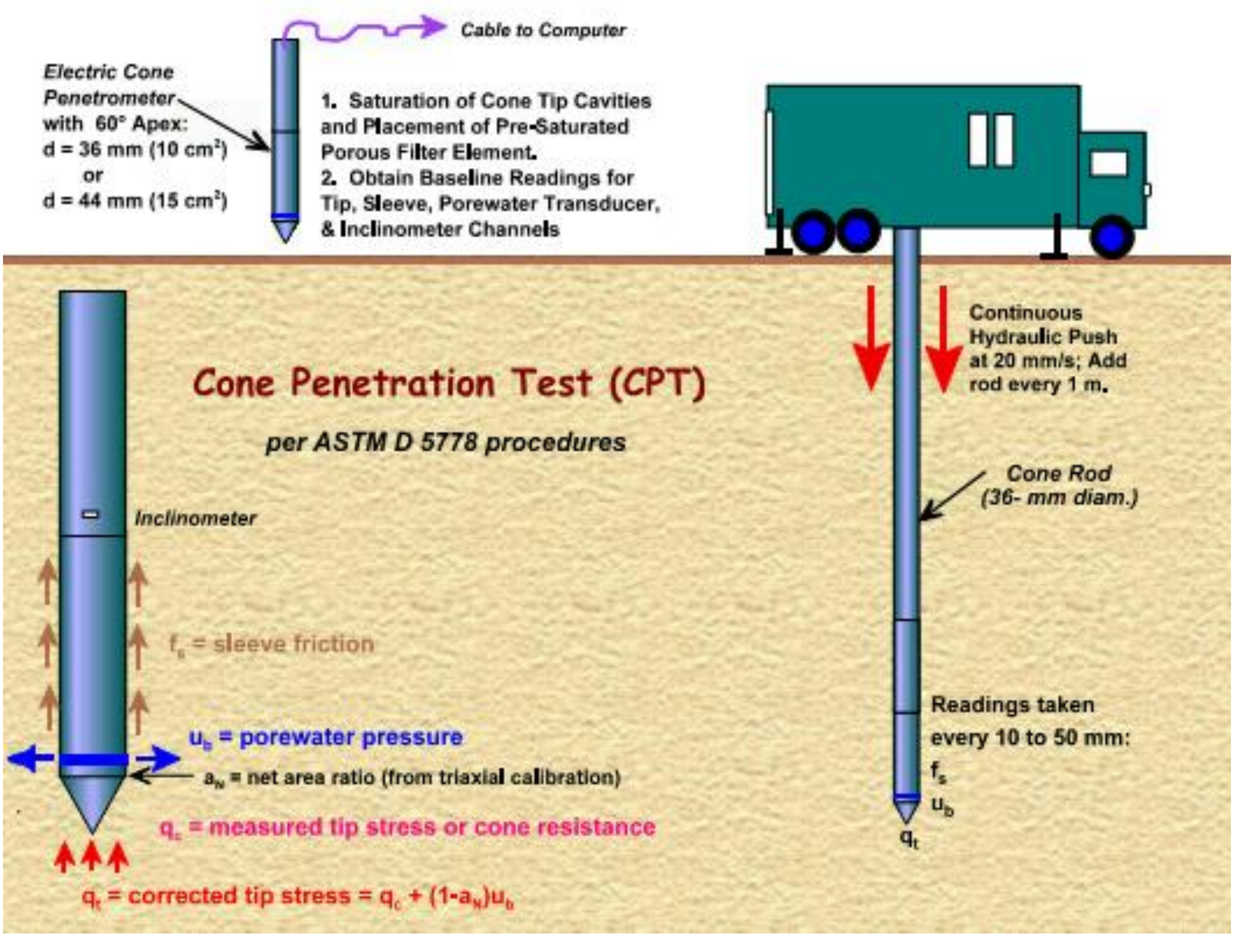


Figure 3 : Cone Penetration Test components and procedures (after FHWA, 2006).

In conducting a CPT, using a developed mechanical cone, initially the inner rod is pushed downwards a distance of 40 mm, causing the cone only to penetrate the soil, and the cone point resistance qc is recorded. The outer shaft is now advanced to the cone base, and the side (or skin) friction resistance q_s is recorded. Now the cone and the engaged friction sleeve are advanced in combination to obtain the total penetration resistance q_T, which should be approximately the sum of the point and side resistances just measured.

In using the electric friction cone, the cone penetration resistance is measured by means of a load cell inside the body of the instrument and can thus be recorded continuously as the penetrometer is pushed into the soil. The results are normally plotted automatically, against depth, by means of a chart recorder. The friction sleeve is mechanically separate from the conical point; side resistance is measured by means of a second load cell. Cone resistance and side resistance can thus be measured independently. A full description of the test apparatus, procedure and results interpretation is given by Meigh (1987). Note: This test has been standardised by ASTM as D-3441.

Types of Cones

Although many different penetrometer styles and configurations have been used, however, the most widely used are the mechanical Dutch cone (Figure 1), and the electric friction cone (Figure 2). The operation of the two types differs in that the mechanical cone is advanced in stages and measures qc and qs at intervals of 20 cm, whereas the electric cone is able to measure qc and qs continuously with depth. In either case, the CPT defines the soil profile with much greater resolution than does the SPT. Another advantage of the CPT is that the disturbance to the soil is minimal.

There are at least five cone types in use, although the ASTM D 3441 standard lists only three.

1. Mechanical—the earliest type, often called the Dutch cone since it was first developed and used in The Netherlands.
2. Electric friction—first modification using strain gauges to measure qc (point resistance) and qs (side friction).
3. Electric piezo—a modification of the electric friction cone to allow measuring the pore water pressure during the test at the cone tip.
4. Electric piezo/friction—a further modification to measure point resistance, sleeve friction, and pore pressure.
5. Seismic cone—a further modification to include a vibration sensor to obtain data to compute the shear wave velocity from a surface hammer impact so that the dynamic shear modulus can be computed [Campanella et al. (1986)].

CPT testing must follow some essential procedures:

- To avoid damaging the cone when penetrating through man-made compacted fills or surficial hard soils, pre-drilling might be necessary.

- The cone thrust direction should be as near as possible to vertical. Its deviation should not exceed 2deg.

- Reference measurements must be obtained, and zero forces on the cone should be recorded at the start and at the end of each CPT sounding.

- The rate of penetration of the cone should range between 10 to 20mm/sec. This implies that a 20m CPT sounding can be completed in about 30min, and excess pore pressures will develop during CPT tests in low permeability soils

- Measurements must be obtained at 25mm to 35mm intervals for a soil under a pavement or for design of pavement depth, or every 150mm to 200mm for any other application. This is the minimum interval prescribed in the standards, more frequent measurements may be obtained with modern equipment.

- During a pause in penetration, any excess pore pressure generated around the cone will start to dissipate. The rate of dissipation depends on the coefficient of consolidation, and thus the permeability of the soil. Dissipation tests can be performed in fine grained soils at any depth to get estimates of permeability, by measuring the decay of excess pore pressures with time.

- When CPTu tests are performed in saturated soft clays and silts, the cone resistance qc must be corrected to account for the pore water pressure acting on the cone geometry as: $qt=qc+u_2(1-a)$ where u_2 is the measured pore water pressure and a is determined from calibration tests (Figure 4). Generally a ranges between a=0.70 to 0.85, and should not be less than a=0.75.

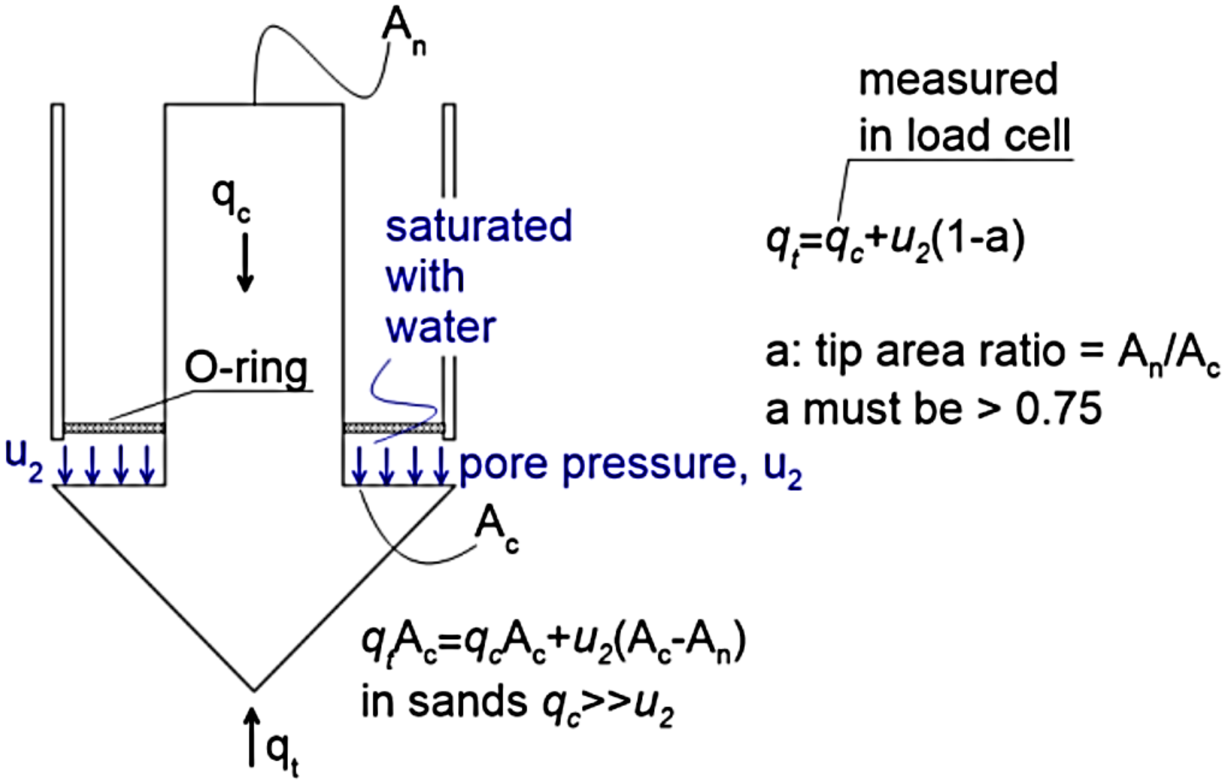


Figure 4: Correction of cone tip resistance for unequal end area effects.

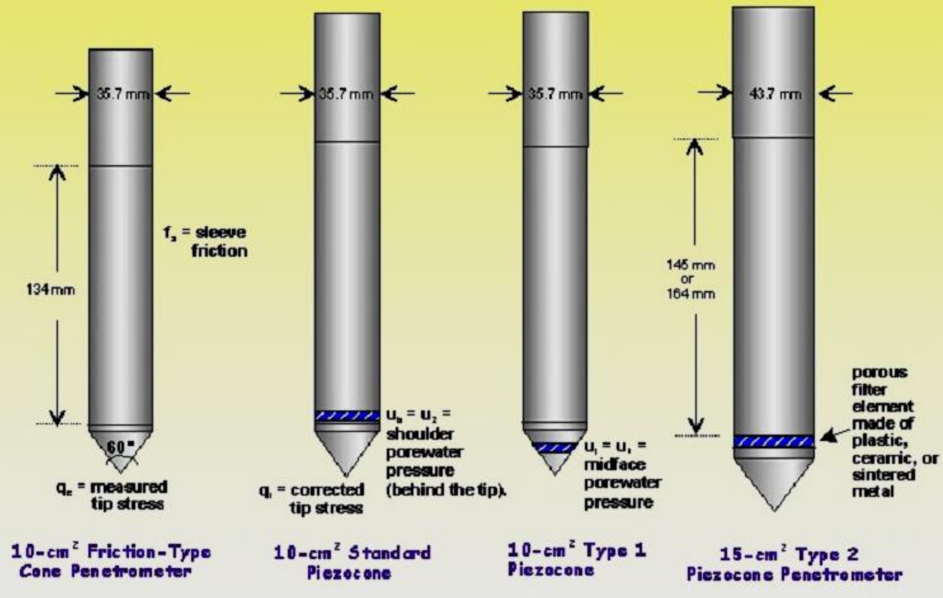
Piezocone

Piezocone is cone penetrometer with added transducers to measure penetration pore water pressures during the advancement.

There are several configurations of the piezocone, and it is critical that the friction sleeve diameter tolerance be 0 to not more than 0.25 mm larger than the cone tip diameter if smaller, the side friction is too low. The piezometer (pore pressure sensor) element may be made from sintered metal, ceramics, or stone. It may be located at the "tip," somewhere along the cone face, or at the cone base and sometimes both in the tip and at the cone base. Both the location of the tip and the type of material to be used in it are important, as any roughness will reflect into the tip resistance. Cone usage in sandy materials quickly roughens the tip.

A serious concern using pore pressure sensors is that they be kept saturated, for any air that is present will substantially reduce the pressure that is recorded. The base location generally produces a lower measured pore pressure than for the tip (or cone face) location.

In sands, the measured penetration pwp are hydrostatic because the high permeability of the sand permits immediate dissipation. While, in clays, the undrained penetration results in the development of excess pwp which may be positive or negative.



Positions of porous tips on piezocones.

Interpretation and Use

Cone tip resistance, q_c can be calculated from:

$$q_c = \frac{F_c}{A_c}$$

where F_c = force required to push the cone into the ground, and A_c plan area of the cone, i.e. 10cm^2.

Local side friction (sleeve friction), f_s is:

$$f_s = \frac{F_s}{A_s}$$

where F_s shear force on the friction sleeve, and A_s = area of the friction sleeve, i.e. 150 cm^2. The friction ratio, f_r (%) is:

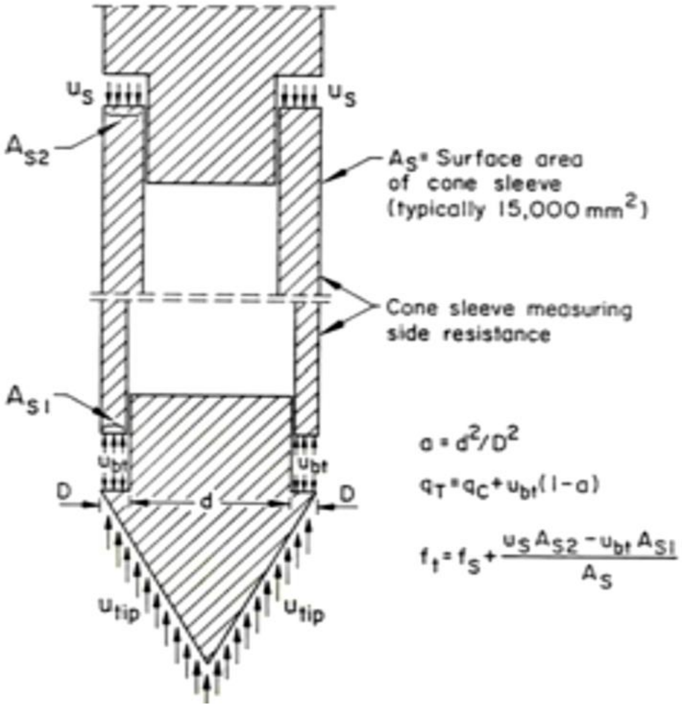
$$f_r = \frac{f_s}{q_c} \times 100$$

where pore water pressure acts downwards on the back of the cone end, the corrected cone resistance, q_t and the corrected sleeve friction, f_t are:

$$q_t = q_c + (1 - \alpha) u_{bt}$$

$$f_t = f_s - \frac{u_s A_{s2} - u_{bt} A_{s1}}{A_s}$$

The value of α is 0.15 to 0.3 for 10 cm^2 cones and 0.65 to 0.8 for 15 cm^2 cones.



Correction detail for pwp acting on cone tip resistance.

In soft cohesive soils, at depth, much of the cone resistance may be derived from the effect of overburden, the ‘net cone resistance, q_n’ is:

$$q_n = q_c - \sigma_v$$

where q_n = net cone resistance, and σ_v = vertical total stress at the level at which q_n is measured.

The friction ratio f_r may also be estimated using D_{50} as in the following empirical quations: Anagnostopoulos et al (2003),

$$f_r(\%) = 1.45 - 1.36 \log D_{50} \text{ (using electric cone)}$$

$$f_r(\%) = 0.781 - 1.611 \log D_{50} \text{ (using mechanical cone)}$$

Note: In developing these equations, the D_{50} of soils ranged from 0.01 mm to about 10 mm.

The friction ratio f_r is primarily used for soil classification as illustrated in the charts of Figures 3 and 4.

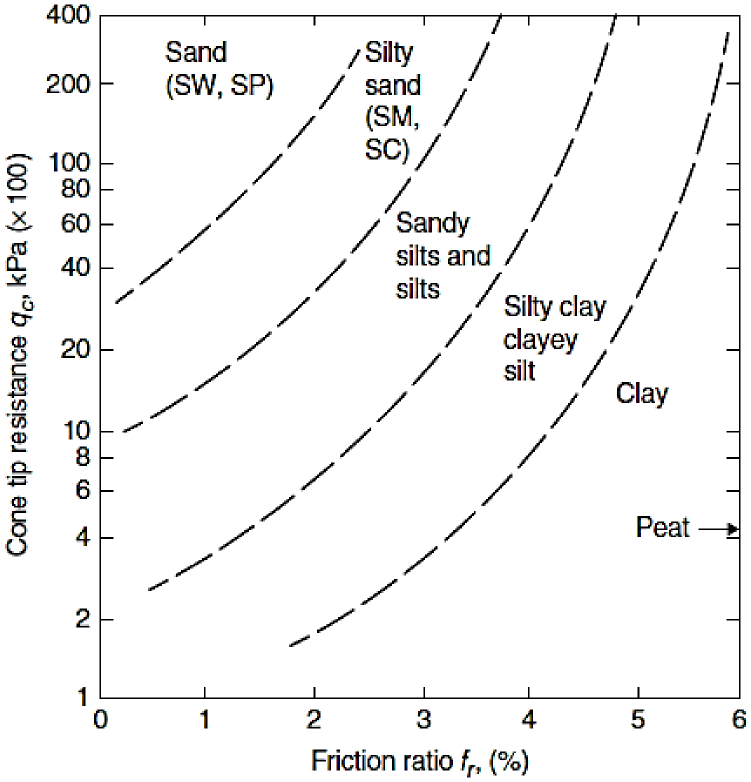
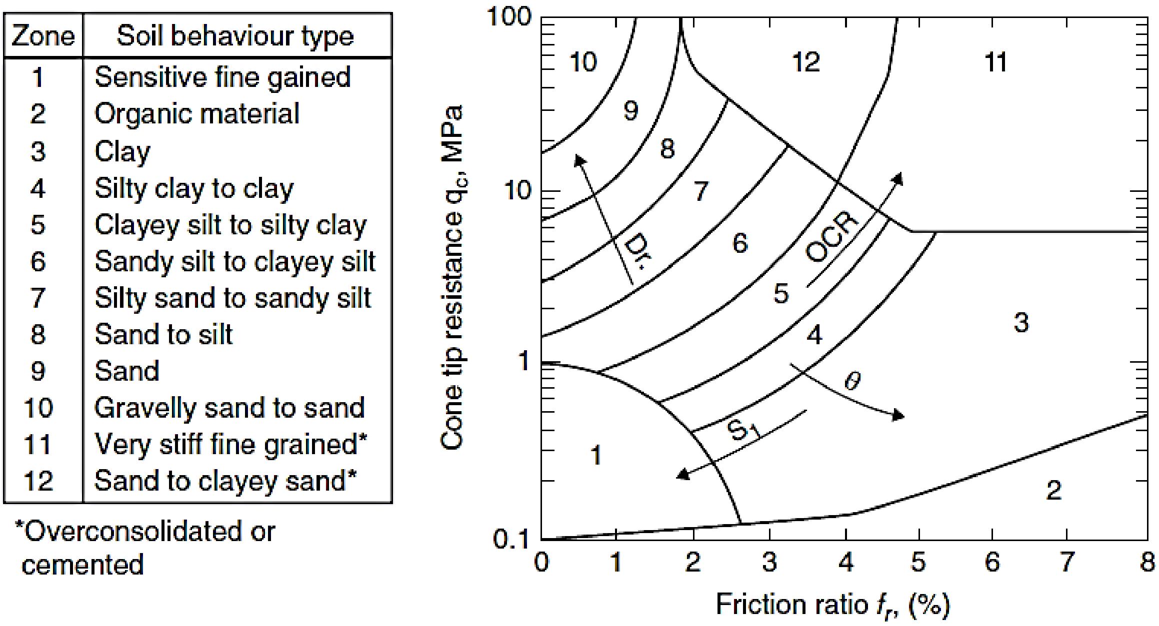


Figure 3: Soil classification chart for standard electric friction or mechanical cone (from Bowles, 2001, after Robertson and Campanella, 1983).



Zone	Soil behaviour type
1	Sensitive fine gained
2	Organic material
3	Clay
4	Silty clay to clay
5	Clayey silt to silty clay
6	Sandy silt to clayey silt
7	Silty sand to sandy silt
8	Sand to silt
9	Sand
10	Gravelly sand to sand
11	Very stiff fine grained*
12	Sand to clayey sand*

*Overconsolidated or cemented

Figure 4: Soil behaviour type chart based on qc and fr (reproduced from paper by P.K. Robertson, 2010; it is the original Robertson and Campanella chart, 1986).

The friction ratio may be used to give an estimate for soil sensitivity S_t as follows (Robertson and Campanella, 1983):

$$S_t \approx \frac{10}{f_r} \qquad f_r \, in \, \%$$

CPT correlations for cohesive soils:

The cone bearing resistance qc (tip bearing resistance) and the undrained shear strength s_u (or c_u) are correlated through the following empirical equation (Mayne and Kemper, 1988):

$$s_u = \frac{q_c - \sigma_0}{N_k}$$

$\sigma_o = \gamma Z$ = overburden pressure where the qc is measured. It is in the same units of qc and same type of pressure (i.e., if qc is an effective pressure, σ'_0 shall be used).

N_k = Cone factor or bearing capacity factor, constant for a certain soil; depends mainly on soil plasticity index (PI) and sensitivity (S_t), and on the cone penetrometer type. Its range is 5–30. The recommended values for mechanical and electric cones are 20 and 15, respectively. Anagnostopoulos et al. (2003) determined these two values equal 18.9 and 17.2, respectively. They also showed that cu equals 0.79 qs and qs for mechanical and electric cones, respectively. For normally consolidated clays of St < 4 and PI < 30, a value of $N_k = 18$ may be satisfactory.

The preconsolidation pressure $\grave{\sigma}_c$ and overconsolidation ratio OCR are correlated as follows (Mayne and Kemper, 1988):

$$\sigma'_c = 0.243(q_c)^{0.96} \qquad (q_c \text{ and } \sigma'_c \text{ are in MN/m}^2)$$

$$OCR = 0.37\left(\frac{q_c - \sigma_o}{\sigma'_o}\right)^{1.01}$$

CPT correlations for cohesionless soils:

Lancellotta (1983) and Jamiolkowsk et al (1985) showed that for the normally consolidated sand, the relative density Dr and the cone resistance qc can be correlated as:

$$D_r(\%) = 66 \times \log \frac{q_c}{\sqrt{\sigma'_o}} - 98$$

where the q_c and σ'_O are in t/m^2

According to Kulhawy and Mayne (1990), the above equation can be rewritten as

$$D_r(\%) = 68\left[\log\left(\frac{q_c}{\sqrt{p_a\,\sigma'_o}}\right) - 1\right]$$

$$P_a = \text{atmospheric pressure} \ (\approx 100\,\text{kPa})$$

Figure 5 represents the empirical relationship recommended by Baldi et al. (1982), and Robertson and Campanella (1983), for normally consolidated quartz sand.

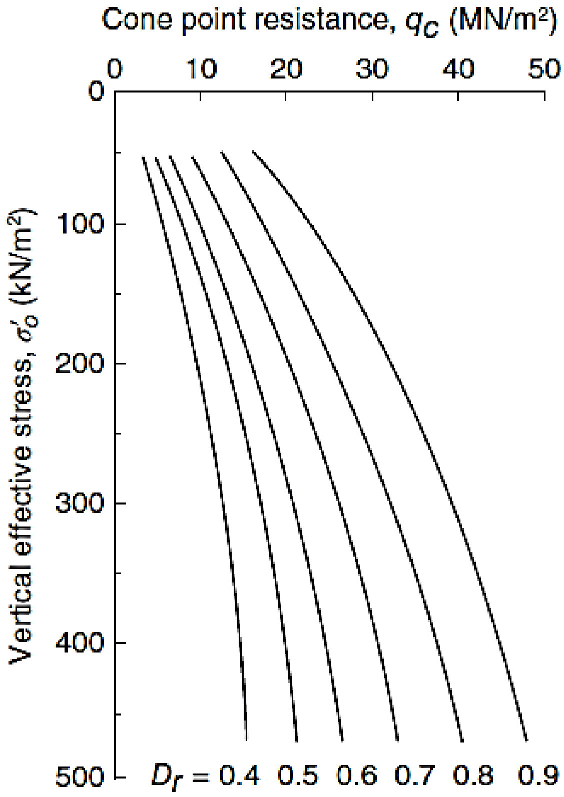


Figure 5: Correlation between qc, Dr and σo for normally consolidated quartz sand (after Baldi et al, 1982; Robertson and Campanella, 1983).

Also, D_r, OCR and q_c are correlated by Kulhawy and Mayne (1990) as

$$D_r = \sqrt{\left(\frac{1}{305 Q_c OCR^{1.8}}\right)\left[\frac{\frac{q_c}{p_a}}{\left(\frac{\sigma'_o}{p_a}\right)^{0.5}}\right]}$$

Where Q_c = compressibility factor; using 0.91, 1.0 and 1.09 for high, moderate and low compressibility of sand, respectively.

For normally consolidated quartz sand, the effective friction angle Ø̀ , σ_O and q_c can be correlated and expressed as (Kulhawy and Mayne, 1990):

$$\varnothing' = \tan^{-1}\left[0.1 + 0.38\log\left(\frac{q_c}{\sigma'_o}\right)\right]$$

Ricceri et al. (2002) suggested a similar correlation for ML and SP-SM soil types as

$$\varnothing' = \tan^{-1}\left[0.38 + 0.27\log\left(\frac{q_c}{\sigma'_o}\right)\right]$$

An approximate correlation (Bowles, 1996) for $\varnothing'$ is

$$\varnothing' = 29^\circ + \sqrt{q_c} + 5^\circ \text{ for gravel;} - 5^\circ \text{ for silty sand, where } q_c \text{ is in MPa}$$

Lee et al. (2004) established a relationship between the horizontal effective stress $\grave{\sigma}_h$,$\grave{\varphi}$ and qc as follows:

$$\varnothing' = 15.575\left(\frac{q_c}{\sigma'_h}\right)^{0.1714}$$

A number of correlations have been proposed to estimate the SPT N-values from CPT results in both cohesive and cohesionless soils. These correlations, generally, use a form of q_c = kN. Unfortunately, all the available correlations cannot be used with much confidence. According to Meyerhof, $q_c \approx 4N_{55}$, where q_c is in kg/cm^2.

Table 1 gives approximate range values of qc/ $N60$ ratio for different soils, using qc in MPa.

Table1 : Approximate range values of qc / $\grave{N}60$ ratio for different soils.

Soil type	q_c/N'_{60}
Silts, sandy silts and slightly cohesive silt-sand mixtures	0.1–0.2
Clean fine to medium sands and slightly silty sands	0.3–0.4
Coarse sands and sands with little gravel	0.5–0.7
Sandy gravels and gravels	0.8–1.0

Anagnostopoulos et al. (2003) proposed the following correlation:

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = 7.6429 D_{50}^{0.26}$$

where p_a = atmospheric pressure (≅100 kPa); q_c in kPa.

Schmertmann (1970) established empirical correlations between E_s and the cone resistance qc. Because CPT can provide a continuous plot of qc versus depth, it is possible to model E_s as a function of depth, which is especially useful. Table 2 presents design range values of E_s/q_c for sands, adapted from Schmertmann et al. (1978), Robertson and Campanella (1989), and other sources.

Table 2: Design range values of Es/qc for sands (from Coduto, 2001).

Soil type	**USCS Group Symbol**	E_s/q_c
Young, normally consolidated clean silica sands*	SW or SP	2.5–3.5
Aged, normally consolidated clean silica sands**	SW or SP	3.5–6.0
Overconsolidated clean silica sands	SW or SP	6.0–10.0
Normally consolidated silty or clayey sand	SM or SC	1.5
Overconsolidated silty or clayey sand	SM or SC	3.0

* Age < 100 years;
** Age > 100 years

Soil Sensitivity

The sensitivity (S_t) of clay is defined as the ratio of undisturbed undrained shear strength to totally remolded undrained shear strength.

$$s_t = \frac{s_u}{s_{u(remolded)}} = \frac{q_t - \sigma_v}{N_{kt}}\left(\frac{1}{f_s}\right)$$

In-Situ Stress Ratio (Ko)

Kulhawy and Mayne (1990) suggested:

$$K_0 = 0.1\left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}}\right)$$

Vane Shear Test (VST)

The difficulty in determining the undrained shear strength (s_u) of very soft and sensitive clays by means of laboratory tests, as a result of the disturbance induced by poor-quality samplers, led to the development of VST.

VST is performed according to BS 1377: Part 9: 1990 and ASTM D2573-72 (Reapproved 1978).
Limit equilibrium analysis is used to relate the measured peak torque to the calculated value of s_u. Both the peak and remolded strengths can be measured; their ratio is termed the sensitivity, S_t.

- VST basically consists of pushing a four-bladed vane, mounted on a solid rod, into the soil and rotating it from the surface.

- VST may be carried out either in the field or in the laboratory. In the field they may be carried out either from ground level, or from the base of a borehole.

- The standard vane has a rectangular geometry with a blade diameter D = 65 mm, height H = 130 mm (H/D =2), and blade thickness e = 2mm is often suitable for soft clays with shear strength up to 50kPa. For stiffer soils, blades of 70 to 100mm in length are large enough to provide good torque resolution.

- By implication, BS 1377 considers that VST is not suitable for testing soils with undrained strengths greater than 75kPa.

Test Procedure

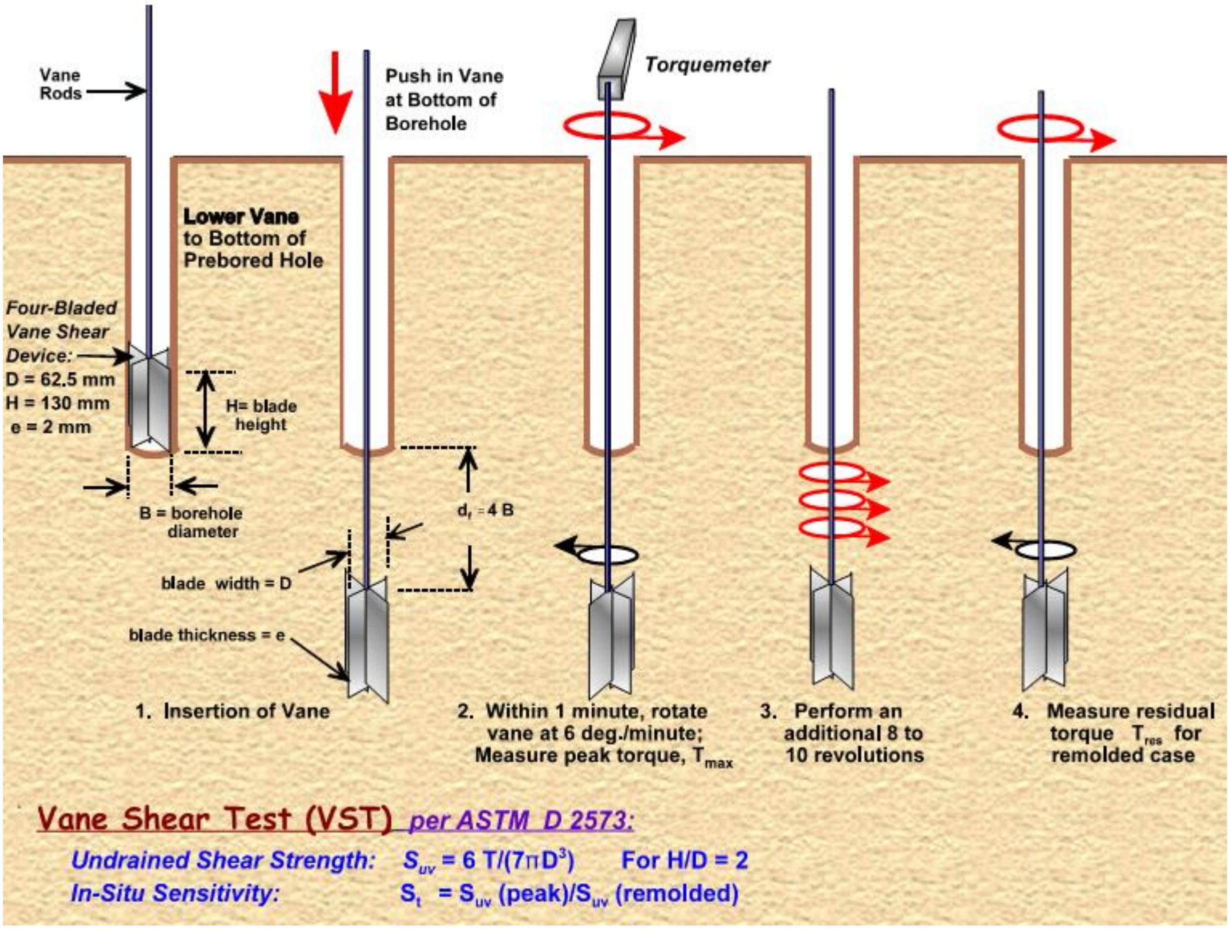
For a borehole of diameter B, the top of the vane should pushed to a depth of insertion of at least d_f = (4-5)B. Within 5 minutes after insertion, rotation should be made at a constant rate.

Two basic penetration probe testing methods are used for land operations:

i. Testing in boreholes: the vane is inserted into the ground from the bottom of a borehole;

ii. Driving inside a protective cover: in this case driving must stop above the test depth with the vane inside the cover.

1. Push the vane slowly with a single thrust from the bottom of the borehole or protected sleeve for the distance required to ensure that it penetrates undisturbed soil.
2. Attach a torque wrench, and turn the rods at a slow but continuous rate of 6-12°/min.
3. Record the relationship between rod rotation (at ground surface) and measured torque by taking readings of both at intervals of (15-30)s. Once maximum torque is achieved, rotate the vane rapidly through a minimum of ten revolutions, and immediately (within 1 mm – ASTM D2573) restart shearing at the previous slow rate, to determine the remoulded strength of the soil.



$$c_u = \frac{2T}{\pi d^2\left(h + \dfrac{d}{3}\right)}$$

where d = width of the vane blade, and h = height of the vane blade

The following equations relate the torque T at failure to the undrained vane shear strength $S_{u,v}$ (or $C_{u,v}$) and vane dimensions:

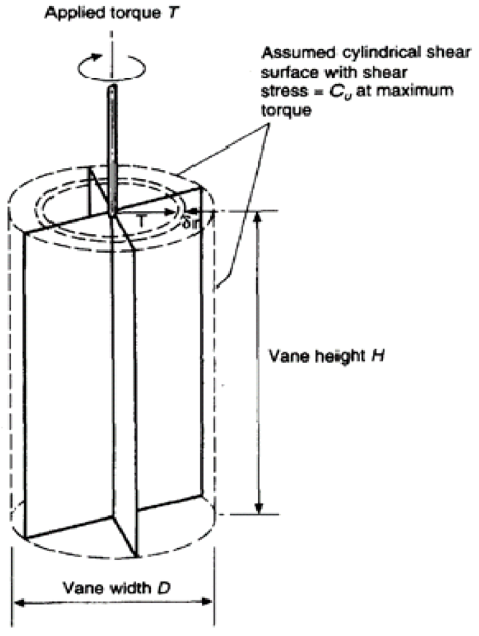
$$T = \pi s_{u,v}\left(\frac{D^2 H}{2} + \frac{D^3}{6}\right)$$

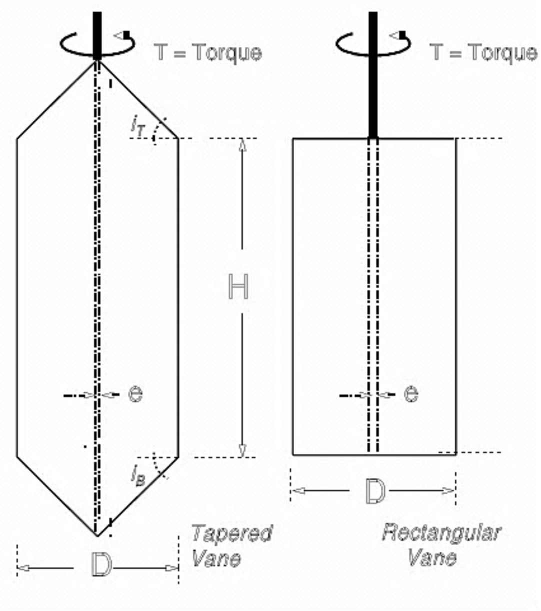
$$T = 7\pi s_{u,v}\left(\frac{D^3}{6}\right) \text{ for } \frac{H}{D} = 2, \qquad s_{u,v} = \frac{6T}{7\pi D^3}$$

Interpretation and Use

The undrained strength is derived on the basis of the following assumptions:

1. penetration of the vane causes negligible disturbance;
2. no drainage occurs before or during shear;
3. the soil is isotropic and homogeneous;
4. the soil fails on a cylindrical shear surface;
5. diameter of shear surface is equal to the width of vane blades;
6. uniform shear stress distribution across the shear surface.





The general expression for all types of vanes (H=2D) and for any end angles is given by:

$$\boldsymbol{s_{uv} = \frac{12T}{\pi D^2[(D/cos i_T) + (D/cos i_B) + 6H]}}$$

where i_T = angle of taper at top (with respect to horizontal) and i_B = angle of bottom taper.

The sensitivity of the soil (S_t) is defined by:

$$\boldsymbol{S_t = \frac{s_{uv(peak)}}{s_{uv(remolded)}}}$$

Clay sensitivity (Skempton and Northey, 1952)

Classification	S_t
Insensitive clays	1.0
Low sensitivity	1-2
Medium sensitivity	2-4
Sensitive	4-8
Extra sensitivity	>8
Quick clays	>16

Corrections

Researchers found that the $S_{u,v}$ values obtained from the FVST are too high for design purposes; its use reduces the factor of safety considerably. It is recommended to use the empirical correction factor λ with $S_{u,v}$ in order to obtain appropriate value for design undrained shear strength. Hence,

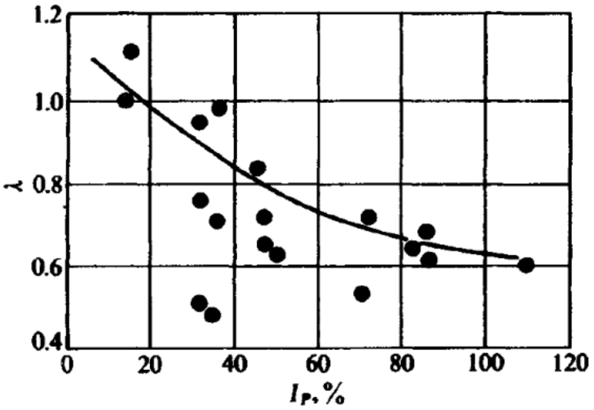
$$\text{Design } s_{u,v} = \lambda \times s_{u,v}$$

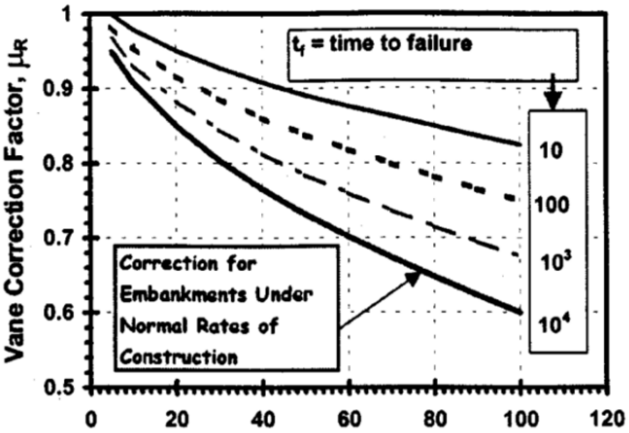
The correction factor λ may be obtained from curves or calculated using any of the following equations: Bjerrum (1972):

$$\lambda = 1.7 - 0.54\log[PI(\%)]$$

PI = plasticity index

Morris and Williams (1994):


1.2
1.0
0.8
0.6
0.4
λ
0
20
40
60
80
100
120
I_P, %


1
0.9
0.8
0.7
0.6
0.5
Vane Correction Factor, μ_R
t_f = time to failure
10
100
10³
10⁴
Correction for Embankments Under Normal Rates of Construction
0
20
40
60
80
100
120