Chapter 5
Cone Penetrometer Test

Abstract This chapter covers one of the most popular insitu tests, the cone penetrometer test (CPT). A brief description of different types of tests has been provided. The piezocone test, an advanced CPT, is described including the test procedure and the parameters obtained in the field. Pore pressure transducer placement locations and the corrections to measured pore pressures are presented. A detailed discussion of soil classification using cone test results is provided. Correlations for design parameters related to sands and clays are discussed separately. For sands, correlations include relative density, friction angle, modulus and small strain modulus. For clays, correlations include the undrained shear strength, over consolidation ratio, constrained modulus, small strain shear modulus, compressibility, friction angle, unit weight and permeability. As the CPT test competes with SPT test for popularity, correlations between the two tests are also discussed. Correlations to use CPT derived parameters directly to calculate the ultimate bearing capacity of shallw and deep foundations are presented. The chapter concludes with a section on liquefaction assessment using CPT as well as SPT results.

Keywords Cone penetrometer test • CPT • Correlations • Piezocone • Soil classification • Liquefaction • Bearing capacity

5.1 Cone Penetrometer Test – General

The cone penetrometer test (CPT) is a versatile in situ test (Fig. 5.1) which has become a routine test for site investigations worldwide to characterize clays and sands. There is little doubt that the cone penetrometer test is one of the the most widely used in situ test in areas where soft and compressible soils occur. As the test is a continuous test, the subsoil profile variation is captured with significantly more details compared to a vane shear test or a SPT which are generally carried out at 1–1.5 m depth intervals. It is a test most useful in weak clays and sands. Latest machinery used for advancing CPT in soils have more power and robustness compared to early equipment and therefore its use in competent soils such as very stiff to hard clays and dense sands is generally not an issue. Modern advances on CPT rigs allow the recovery of undisturbed samples or carry out vane shear tests in addition to carrying out conventional CPT testing. This is very advantageous

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Fig. 5.1 CPT machine in mud flats (Courtesy Yvo Keulemans, CPTS)

because of significant additional costs involved if a separate borehole rig has to be mobilised for sampling and vane shear testing.

The main disadvantage of the CPT is that it does not provide an absolute value for soil parameters and the results need to be calibrated against other tests such as vane shear and laboratory tests such as triaxial tests. Where such data are not available, practitioners use local experience and/or empirical values to derive design parameters

There are generally three main types of cone penetrometers:

1. Mechanical cone penetrometer – Also known as the Dutch Cone Penetrometer or the Static Cone Penetrometer, this uses a set of solid rods or thick walled tubes to operate the penetrometer. The penetrometer tip is initially pushed about 4 cm and the tip resistance is recorded. Then both cone and sleeve are pushed together to record the combined tip and cone resistance. This is repeated with depth to provide a profile for cone and sleeve resistances. The procedure allows a measurement be taken at about every 20 cm.
2. Electric cone penetrometer – An advancement of the mechanical cone, the electric cone has transducers to record the tip and sleeve resistances separately. Therefore it has the advantage of advancing the cone continuously to obtain a continuous resistance profile and the inner rods are not required.
3. Electric cone penetrometer with pore pressure measurements (Piezocone) – A further addition to the penetrometer is the inclusion of pore pressure transducers at the tip or on the sleeve to record continuous pore pressure measurements. The test is widely known with the abbreviation CPTu. A more popular name for the CPTu equipment is the piezocone. In Chap. 1, Table 1.1, where applicability and usefulness of in situ tests are summarised, it is evident that the piezocone has the best rating amongst in situ tests for parameters obtained or the ground type investigated.

A further development of penetrometer type is the seismic cone penetrometer test which allows the measurement of the shear wave velocity with depth. The equipment consists of the piezocone unit plus a receiver for seismic measurements. Generally, at every 1 m interval (i.e. at rod breaks), a shear wave is generated at the ground surface and the seismic wave arrival time is recorded. The shear wave velocity could be converted to a shear modulus (see Sect. 5.5.4) using empirical correlations. The seismic cone penetrometer test is not addressed here and the reader is referred to Mayne (2007).

The main difference between the traditional CPT and the CPTu is the measurement of pore pressure in the latter test. The measurement of pore pressure provides a wealth of data, with a pore pressure profile at the test location, reflecting the different soil types. This type of data was never available to the designer as no other test in history could measure continuous pore pressure with depth. CPTu therefore became very popular within a relatively short time although traditional CPTs are still in use, probably because of the cost of new equipment and accessories. Most likely, the traditional CPT will be phased out from the market, especially in the developed world, because of the advantages offered by the CPTu.

Equations and empirical relationships available for the CPT are equally valid for the CPTu with additional relationships established due to extra information provided by pore pressure measurements. Therefore, in this chapter, we will be referring to the CPTu rather than the CPT although correlations that do not include pore pressure measurements would still be valid for both.

One of the other achievements in the CPTu is its ability to carry out dissipation tests to obtain the coefficient of consolidation of clays. At a nominated depth, advancement of the cone is stopped and the pore pressure generated is allowed to dissipate and the measurements are recorded continuously. Generally a target of 50 % dissipation is adopted because of time constraints. Even such a limited duration in soft soils could be in the order of an hour or two. Where the dissipation is very slow because the coefficient of consolidation, c_v, is very low, some operators leave the test overnight for dissipation. Readers are referred to Lunne et al. (1997) for a description of the test and derivation of geotechnical parameters related to the rate of consolidation.

5.2 Piezocone Test – Equipment and Procedure

ISSMGE Technical Committee 16 (TC16) (Ground Characterization from In Situ Testing) published an International Test Procedure for the CPT and the CPTu. The information given below on the equipment (see Fig. 5.2) and procedure is mostly based on that report.

The piezocone test consists of a cone and a surface sleeve continuously pushed into the ground and the resistance offered by the cone and sleeve measured

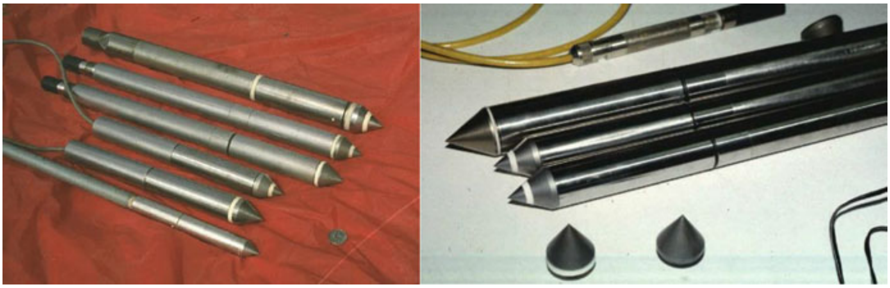


Fig. 5.2 Various Cone Penetrometers including Electric Friction and Piezocone types (After FHWA NHI-01-031 – Mayne et al. 2002)

electronically, in addition to measuring the pore pressure by the use of a pore pressure transducer. The standard cone tip is usually 10–15 cm^2 and has an apex angle of 60°). The cone is pushed with a standard rate of penetration of 20 ± 5 mm/s. Figure 5.3 shows the location of main components i.e. cone, sleeve and the pore pressure transducer of the probe which is pushed down by rods. The measurements taken with depth include:

- Tip resistance (q_c)
- Sleeve friction (f_s)
- Pore water pressure (u) – could be measured at the cone face (u_1), shoulder (u_2) or top of the sleeve (u_3) (see Fig. 5.3)

Tip resistance (q_c) is obtained by measuring the ultimate force (Q_c) experienced by the cone only divided by the area of the cone (A_c).

$$q_c = \frac{Q_c}{A_c} \tag{5.1}$$

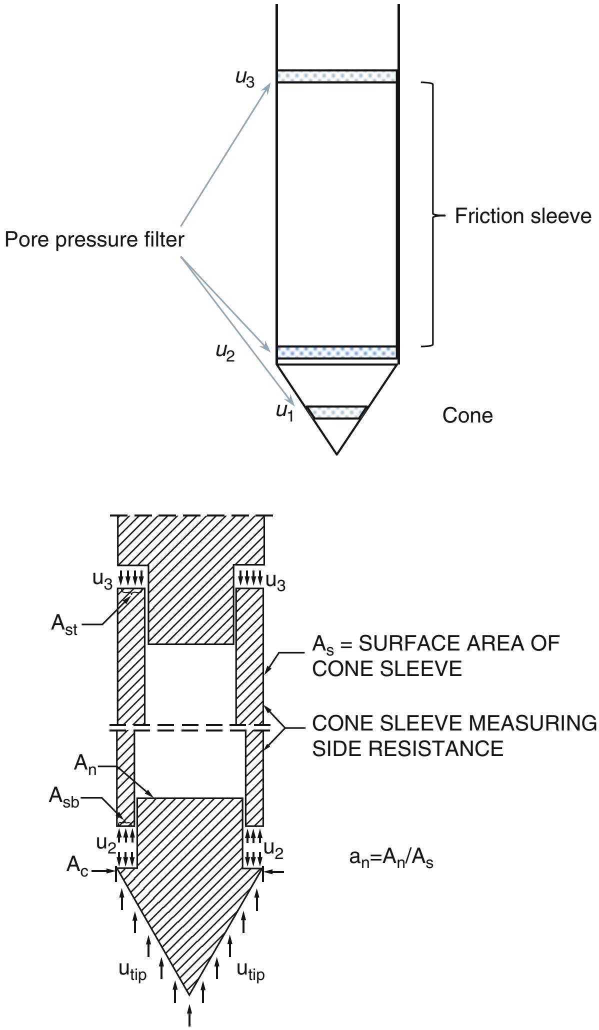
Sleeve friction (f_s) is obtained by measuring the ultimate force (Q_s) only on the sleeve, i.e., side friction, divided by the area of the sleeve (A_s).

$$f_s = \frac{Q_s}{A_s} \tag{5.2}$$

In piezocones/CPTu's, the pore pressure transducer is located generally at mid-face of the cone (measuring u_1) or at the shoulder (measuring u_2) i.e. where cone and sleeve meet (see Fig. 5.2). The resistance measurements are influenced by the water pressure acting behind the cone tip and the edge of the sleeve. Therefore a correction needs to be applied.

When the pore pressure transducer is located at the shoulder, the following equation could be used to correct the cone resistance (Jamiolkowski et al. 1985; Campanella and Robertson 1988; Lunne et al. 1997; Campanella et al. 1982; Mayne 2007):

Fig. 5.3 Components and correction details of a piezocone (Adapted from FHWA NH1-01-031 – Mayne et al. 2002)



$$q_t = q_c + u_2\,(1 - a_n) \tag{5.3}$$

where

q_t = corrected tip resistance
u_2 = pore pressure measured at the shoulder
a_n = net area ratio, approximately equal to the ratio of shaft cross section and cone cross section areas (usually measured in a calibration cell, Lunne et al. 1997)

As Lunne et al. (1997) points out, although a value close to 1.0 is ideal, the ratio a_n generally ranges from 0.55–0.9. However, values as low as 0.38 have been recorded which should be unacceptable when the test is carried out in very soft fine grained soils because the correction becomes the main contribution to q_t.

A correction factor is also applicable to the sleeve friction (Lunne et al. 1997):

$$f_t = f_s + \frac{(u_3 A_{st} - u_2 A_{sb})}{A_s} \tag{5.4}$$

where

A_{sb} = cross sectional area of the sleeve at the base – Fig. 5.3.
A_{st} = cross sectional area of the sleeve at the top – Fig. 5.3.
A_s = surface area of the sleeve – Fig. 5.3.

In clayey soils, the magnitude of pore water pressures generated during a test could be high and the correction can be significant. Therefore, the correction factor is most important to correct the recorded tip resistance and provide more accurate results. However, pore water pressure correction is not important for sands because the pore pressure generated is not significant and therefore the measured pore pressure purely reflects the height of the groundwater table i.e, measures the hydrostatic pressure. Therefore it is not significant whether q_t or q_c is used for engineering assessments when sandy soils are present.

Although the ISSMGE TC16 (1999) Reference Test Procedure refers to pore pressure measurement at the shoulder, i.e. u_2, some penetrometers measure the pore pressure at the cone (u_1) in which case u_2 could be obtained as follows (Lunne et al. 1997):

$$(u_2 - u_0) = K(u_1 - u_0) \tag{5.5}$$

where u_0 = equilibrium pore pressure (due to groundwater table)

Typical values for K presented by Lunne et al. 1997 (modified after Sandevan 1990) are presented in Table 5.1.

Two other important parameters related to CPTu and will be discussed later are the Friction Ratio (R_f) and the pore water pressure parameter (B_q). R_f is deduced from the two parameters q_t and f_t:

$$R_f = \frac{f_t}{q_t} \tag{5.6}$$

B_q is derived from the following equation:

$$B_q = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \tag{5.7}$$

where

σ_{vo} = total overburden stress

Table 5.1 Typical values for adjustment factor *K* if filter is located at the cone (Lunne et al. 1997)

Soil type	*K*	u_1/u_0
CLAY normally consolidated	0.6–0.8	2–3
CLAY slightly overconsolidated, sensitive	0.5–0.7	6–9
CLAY heavily overconsolidated, stiff	0–0.3	10–12
SILT loose, compressible	0.5–0.6	3–5
SILT dense, dilative	0–0.2	3–5
SAND loose, silty	0.2–0.4	2–3

5.3 Practical Use of Penetrometer Test Results

The cone penetrometer test is a complex test to be analysed and it is largely used with empirical relationships for all practical purposes. In addition to soil classification and provision of a continuous profile the test allows the derivation of several geotechnical parameters. The main uses of the test for the practitioners could be summarized as follows:

1. Soil classification
2. Correlations for Cohesionless soils
   (a) Relative density
   (b) Friction angle
   (c) Modulus
   (d) Small strain shear modulus
3. Correlations for Cohesive soils
   (a) Undrained Shear Strength
   (b) Sensitivity
   (c) Over consolidation ratio (*OCR*)
   (d) Modulus and compressibility
   (e) Small strain shear modulus
   (f) Friction angle
4. Correlation with unit weight
5. Correlation with foundation resistance
6. Correlation with SPT
7. Correlation with permeability

5.4 Soil Classification

One of the primary objectives of a cone penetrometer test is to identify the soil profile from the test results. While empirical rules have been established for this purpose as discussed later, it should be stressed that a probe test can never displace/replace borehole sampling which allows physical observation of the materials and allows laboratory tests to be carried out. Therefore it is generally considered good practice to conduct boreholes to supplement the CPT programme so that CPT results could be calibrated against and also to collect samples for laboratory testing.

Initially, calibration of penetrometer test results were carried out using the cone resistance and friction ratio as pore pressure measurements were not available until the advent of the piezocone. One of the earliest comprehensive classifications was carried out by Douglas and Olsen (1981) using cone resistance and the friction ratio. It is generally accepted that the measurement of sleeve friction is often less accurate and less reliable than the cone resistance.

The advent of the piezocone allowed the additional measurement of pore pressure which allows better classification of soils. The main additional parameter used for soil classification is the pore pressure ratio B_q [Eq. (5.7)].

Over the years, many an author has proposed classification methods and charts based on either CPT or CPTu test results. They include Schmertmann (1978), Robertson et al. (1986), Robertson (1990), Eslami and Fellenius (1997), Olsen and Mitchell (1995), Senneset et al. (1989), Jones and Rust (1982), Ramsey (2002) and Jefferies and Davies (1991). Long (2008) reviewed the proceedings of various conferences to find out the commonly used classification charts by academics, researchers and practitioners. He concluded that, over the period 1998–2006, Robertson et al. (1986) and Robertson (1990) charts are the most popular. Long (2008) also reports on Molle (2005) who carried out a review of published literature to determine the reliability of different classification charts and concluded that Robertson et al. (1986) and Robertson (1990) charts provided reasonable to very good results. Therefore only these two methods are briefly discussed below.

The two methods of Robertson et al. (1986) and Robertson (1990) use the following parameters for the classification charts:

- Robertson et al. (1986) – q_t, B_q and R_f (Fig. 5.4)
- Robertson (1990) – Q_t (normalized q_t), B_q and F_r (normalized friction ratio) (Fig. 5.5)

where

$$\text{Normalized cone resistance}\, Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \tag{5.8}$$

$$\text{Normalized friction ratio}\, F_r = \frac{f_s}{q_t - \sigma_{v0}} 100\,(\%) \tag{5.9}$$

$$\text{effective vertical stress}\, \sigma'_{vo} = \sigma_{vo} - u_0 \tag{5.10}$$

From the two charts in Fig. 5.5, only the first chart could be used if a CPT is carried out without pore pressure measurements.

Robertson et al. (1986) state that linear normalization of cone resistance (Q_t) is best suited for clay soils and less appropriate for sands.

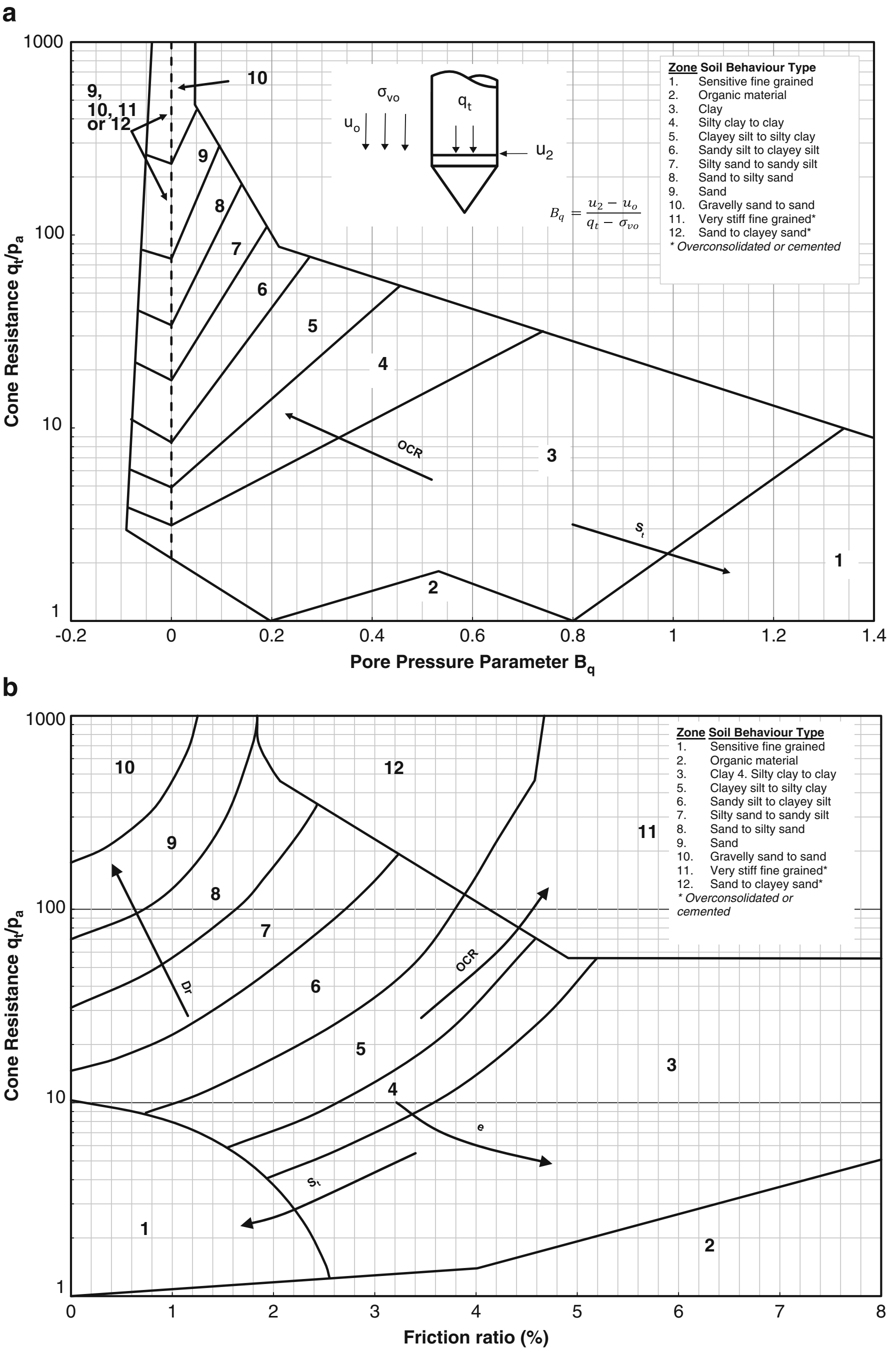
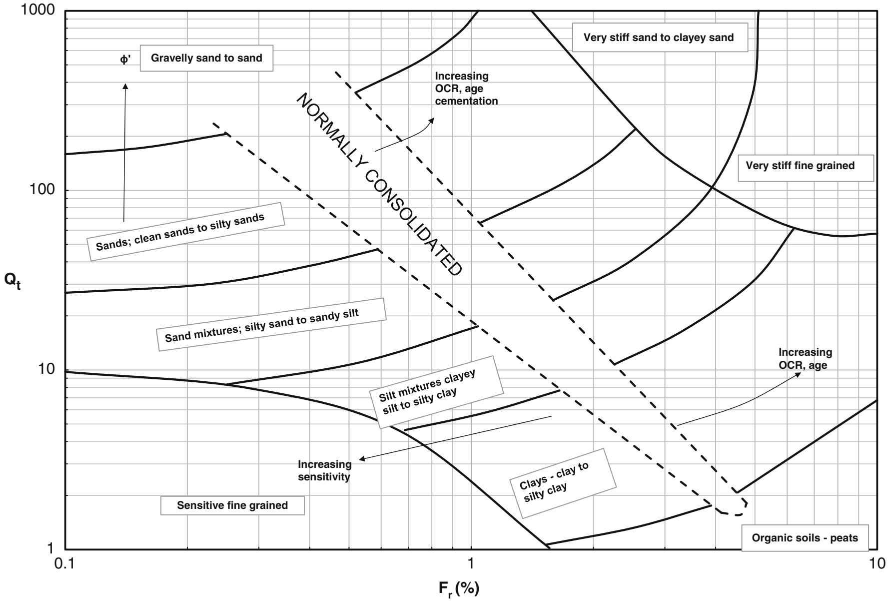


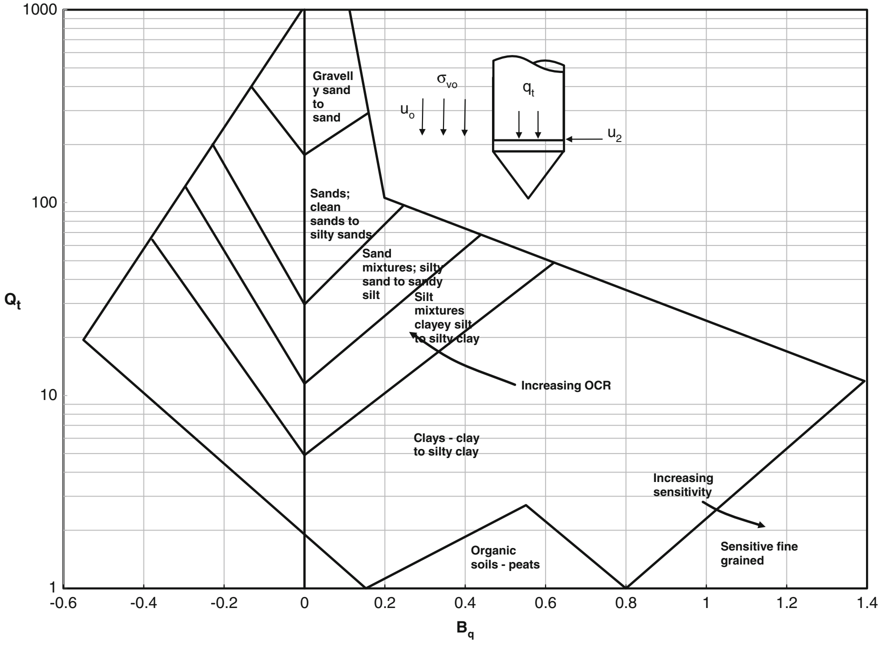
Fig. 5.4 Proposed soil behaviour type classification system from CPTu data (Adapted from Robertson et al. 1986)

a



$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad B_c = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \qquad F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

b



$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \qquad B_c = \frac{u_2 - u_o}{q_t - \sigma_{vo}} \qquad F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$$

Fig. 5.5 Soil behaviour type classification chart based on normalised CPT/CPTu data (adapted from Robertson 1990)

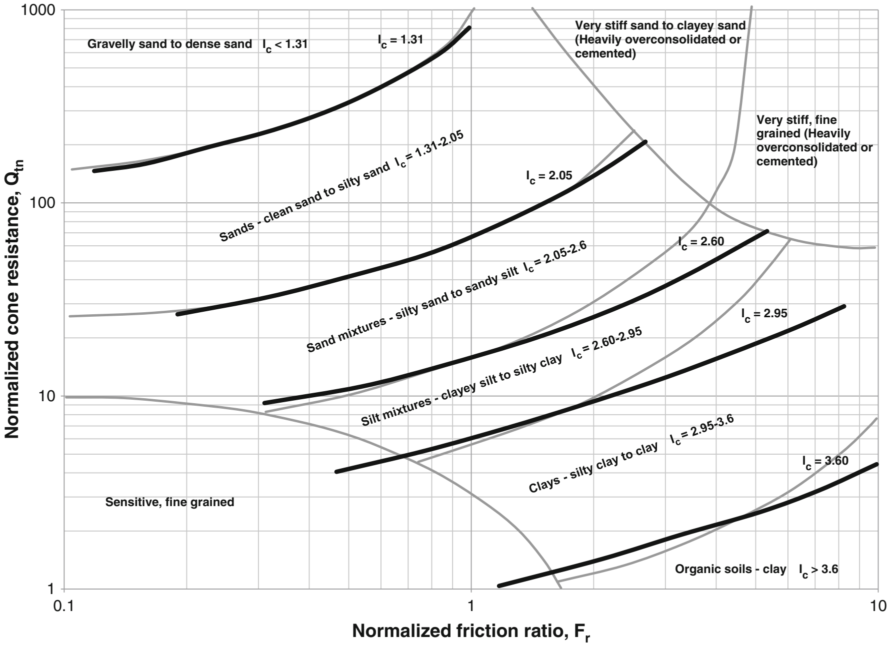


Fig. 5.6 Contours of SBT index, I_c on normalized SBT charts of Robertson (1990)
Note – Normally Consolidated Zone (see Fig. 5.5) left out for clarity but should be superimposed

Jefferies and Davies (1993) introduced an index I_c to represent Soil Behaviour Type (SBT) zones in the Q_t vs F_r chart (Fig. 5.5) of Robertson (1990). The index I_c is the radius of circle defining the zone boundaries. Robertson and Wride (1998) presented a modification to this index that could be applied to the chart in Fig. 5.5. The modified definition is:

$$I_c = \left[(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2\right]^{0.5} \tag{5.11}$$

Figure 5.6 shows the modified chart of Fig. 5.5.

5.5 Correlations for Sands

5.5.1 Correlation with Relative Density of Sand

Relative density, D_r, is a parameter used in sands to identify the level of compaction of the material (see Sect. 2.3.2). It is given by the formula:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{5.12}$$

where

e_{max} = maximum void ratio
e_{min} = minimum void ratio
e = in situ void ratio

This equation can be re-written as follows:

$$D_r = \frac{\left(\rho_d - \rho_{d\,min}\right)}{\left(\rho_{d\,max} - \gamma_{d\,min}\right)} \frac{\rho_{d\,max}}{\rho_d} \tag{5.13}$$

where

$\rho_{d\ max}$ maximum dry density
$\rho_{d\ min}$ minimum dry density
ρ_d in situ dry density

Relative Density could be evaluated using laboratory and field tests specifically catered to obtain density values:

1. Use laboratory procedures to find minimum and maximum density, and e_{max} and e_{min}
2. Use in situ tests such as sand replacement method and nuclear gauge test to find the in situ dry density and hence e.

There are various country standards detailing the laboratory procedures to obtain the maximum and minimum density including ASTM Standards D4253-14 and D4254-14. However, the in situ density using sand replacement or nuclear gauge can only be measured at shallow depth because it is not practical or feasible to excavate test pits more than about 1 m deep to carry out such tests. The use of CPT and SPT type tests are used widely in the industry to assess relative density by the use of empirical rules to convert the penetration resistance.

Early research has categorized the density to describe the relative behaviour as shown in Table 5.2.

Baldi et al. (1986) presented several correlations for both normally consolidated (NC) and over consolidated (OC) sands. The following correlation is for NC sand:

$$D_r = \left(\frac{1}{C_2}\right) \ln\left(\frac{q_c}{C_0\left(\sigma'_{vo}\right)^{0.55}}\right) \tag{5.14}$$

where:

C_0 and C_2 = soil constants (For moderately compressible, normally consolidated, unaged and uncemented, predominantly quartz sands, C_0 = 157 and C_2 = 2.41)

Table 5.2 Borderline values of D_r, N and ϕ' for granular soils

^Very loose | Loose | Medium dense | Dense | Very dense

#D_r (%)	0	15	35	65	85	100
*N_{60}		4	10	30	50	
##$(N_1)_{60}$		3	8	25	42	
**ϕ'(deg)		28	30	36	41	
##$(N_1)_{60}/D_r^2$			65	59	58	

'Terzaghi and Peck (1948); #Gibbs and Holtz (1957); ##Skempton (1986); ''Peck et al. (1974)

q_c = cone penetration resistance in kPa
σ'_{vo} = effective vertical stress in kPa

Mayne (2007) states that most correlations in the 70's and 80's did not consider the boundary effects of the calibration chambers and refers to Jamiolkowski et al. (2001) who introduced a correction factor in reexamining previous results and the expression proposed was as follows:

$$D_r = 100\left[0.268\ln\left(\frac{q_t/p_a}{\sqrt{\sigma'_{vo}/p_a}}\right) - 0.675\right] \quad (5.15)$$

Another widely used relationship was proposed by Kulhawy and Mayne (1990) who highlighted the effects of compressibility and over consolidation ratio (OCR) on the relationship between the relative density and the dimensionless cone resistance. Based on available corrected calibration studies they proposed the following approximate solution to capture the different relationships:

$$D_r^2 = \frac{Q_{cn}}{305\,Q_c\,Q_{OCR}} \quad (5.16)$$

where

$Q_{cn} = (q_c\,/\,p_a)\,/\,(\sigma'_v\,/\,p_a)^{0.5}$
Q_C = Compressibility factor (0.91 for high, 1.0 for medium and 1.09 for low). Kulhawy and Mayne (1990) state that majority of the natural sands are likely to be of medium to high compressibility
Q_{OCR} = Overconsolidation factor = $OCR^{0.18}$
p_a = Atmospheric pressure in same units as q_c
σ'_v = effective vertical stress in same units as q_c

Table 5.3 Values of coefficients *a* and *c* in Eq. (5.19)

Reference	*a*	*c*	Remarks
Burland and Burbidge (1985)	0.33	15.49	Upper limit
	0.32	4.90	Lower limit
Robertson and Campanella (1983)	0.26	10	Upper limit
	0.31	5.75	Lower limit
Kulhawy and Mayne (1990)	0.25	5.44	
Canadian Geotechnical Society (1992)	0.33	8.49	
Anagnostopoulos et al. (2003)	0.26	7.64	

The above equation is similar in form to the equation Kulhawy and Mayne (1990) proposed for the SPT with the most important difference being SPT relationship included aging whereas the above CPT relationship is only for unaged sands. They suggested that, if the same functional relationship for aging holds for both the SPT and CPT, then it is necessary to include the aging factor they proposed for the SPT relationship. This results in the following equation although they warn that this addition is currently speculative:

$$D_r^2 = \frac{Q_{cn}}{305\, Q_c\, Q_{OCR}\, Q_A} \tag{5.17}$$

where

$$Q_A = \text{Aging factor} = 1.2 + 0.05 \log(t/100),\ t \,\text{in years} \tag{5.18}$$

Das and Sivakugan (2011) summarise the work of several authors who investigated the relationship between q_c, N_{60} (SPT at 60 % efficiency) and D_{50} (median grain size). The correlations can be expressed as follows:

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = c D_{50}^a \tag{5.19}$$

Table 5.3 lists the average values for *a* and *c* from these studies.

Further relationships between SPT *N* and q_c are presented in Sect. 5.9.

5.5.2 *Correlation of* q_c *with Sand Friction Angle,* ϕ'

There have been several attempts to interpret the friction angle of sand from CPT, specifically the CPT tip resistance, q_c (Janbu and Senneset 1974; Durgunoglu and Mitchell 1975; Villet and Mitchell 1981). One of the earliest contributions was by Meyerhoff (1956) who presented the following Table 5.4 based on the Static Cone Penetrometer.

Table 5.4 Correlation of q_c and relative density with friction angle for cohesionless soils (After Meyerhoff 1956)

State of packing	Relative Density	*SPT N*	q_c in MPa	Approximate triaxial friction angle (degrees)
Very loose	<0.2	<4	<2	<30
loose	0.2–0.4	4–10	2–4	30–35
Medium dense	0.4–0.6	10–30	4–12	35–40
dense	0.6–0.8	30–50	12–20	40–45
very dense	>0.8	>50	>20	>45

Table 5.5 Correlation of q_c and relative density with friction angle for cohesionless or mixed soils (After Bergdahl et al. 1993)

Relative density	q_c (MPa)	φ' *(degrees)*
very weak	0.0–2.5	29–32
weak	2.5–5.0	32–35
medium	5.0–10.0	35–37
large	10.0–20.0	37–40
very large	>20.0	40–42

Robertson and Campanella (1983) proposed an empirical relationship to be applicable to uncemented, unaged, moderately compressible quartz sands after reviewing calibration chamber test results and comparing with peak friction angle from drained triaxial tests. The relationship was presented as a graph of log (q_c/σ'_{vo}) against tan ϕ'. The design chart has been approximated to the following (Robertson and Cabal 2012):

$$\tan \phi' = \frac{1}{2.68}\left[log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right] \tag{5.20}$$

Dysli and Steiner (2011) cite the contribution by Bergdahl et al. (1993) as shown in Table 5.5 correlating the friction angle with q_c and relative density, for granular soils.

Mayne (2007) cites the following correlation based on the results obtained in calibration chambers:

$$\phi' = 17.6 + 11.0\log(q_{t1}) \tag{5.21}$$

where

$$q_{t1} = \frac{(q_{ct}/p_a)}{(\sigma'_{v0}/p_a)^{0.5}} \tag{5.22}$$

5.5.3 Correlation with Constrained Modulus of Cohesionless Soils

As previously discussed, to obtain undisturbed samples in cohesionless soils is difficult and expensive. Therefore, practitioners favour the use of in situ testing to determine the deformation properties by the use of empirical correlations. At shallow depth, tests such as the plate load test provide a simple but effective test although this test cannot be used to assess the deeper profile. Trial embankments could provide good evaluation of the modulus along the depth profile if adequate instrumentation such as settlement plates and extensometers are used. Obviously such tests are very expensive and may not be justified when only a limited budget is available for the site investigation. CPT, presuremeter test and dilatometer test therefore become an important tool available to the geotechnical designer.

As Baldi et al. (1989) state, although the stiffness of cohesionless soils depends on many factors including the grading, mineralogy, angularity, grain fabric, stress-strain history, mean effective stress, drainage conditions etc., in a given soil, penetration resistance is primarily controlled by the void ratio/relative density and the state of the effective stress. Based on a large number of tests carried out in situ and in a calibration chamber, proposed correlations to obtain the drained Young's modulus of silica sands based on cone penetration resistance is shown in Fig. 5.7 (Bellotti et al. 1989).

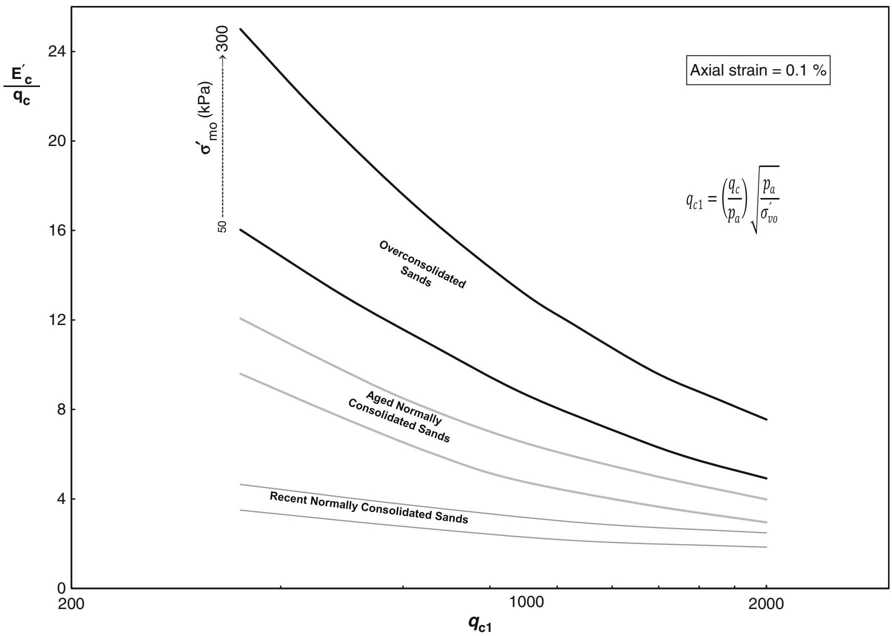


Fig. 5.7 Evaluation of drained Young's modulus from CPT for silica sands (Adapted from Bellotti et al. 1989)

Table 5.6 Initial tangent constrained modulus correlation with q_c (Reported by Lunne et al. 1997)

	Constrained modulus (M_o) relationship with q_c (MPa)	*Applicable q_c range*
NC Unaged	$M_o = 4\ q_c$	q_c <10 MPa
	$M_o = 2\ q_c + 20$ (MPa)	10MPa < q_c <50 MPa
	$M_o = 120$ MPa	q_c >50 MPa
OC	$M_o = 5\ q_c$	q_c <50 MPa
	$M_o = 250$ MPa	q_c >50 MPa

Note – M_o value represents the modulus at the instu effective vertical stress, σ'_{vo}, before the start of in situ test

As Lunne et al. (1997) points out, most correlations between penetrometer test results and the drained constrained modulus refer to the tangent modulus as found from oedometer tests. The correlations are typically represented as:

$$\mathrm{M} = \alpha q_c \tag{5.23}$$

where M = Constrained modulus and α = constrained modulus factor, a constant

Robertson and Campanella (1983), summarizing the work of Lunne and Kleven (1981) who reviewed calibration chamber results of different authors, commented that the results indicate an α of 3 should provide the most conservative estimate of 1-D settlement and that the choice of α value depends on judgment and experience.

Lunne et al. (1997) presented the work of Lunne and Christophersen (1983) for in situ tangent modulus (M_0) for unaged, uncemented, predominantly silica sands (see Table 5.6). As noted by Lunne et al. (1997), these correlations were based on tests carried out to a level of axial strain equal to 0.1 %, corresponding to the upper limit of the average vertical strain of practical interest of shallow and deep foundations in cohesionless soils.

To calculate modulus for higher stress ranges, Lunne and Christophersen (1983) recommended Janbu's (1963) formulation (see Lunne et al. 1997):

$$M = M_o \sqrt{\frac{\left(\sigma'_{vo} + \frac{\Delta\sigma_v}{2}\right)}{\sigma'_{vo}}} \tag{5.24}$$

where

$\Delta\sigma'_{vo}$ = additional stress above the initial stress

Robertson and Cabal (2014) suggested the following formulation:

$$M = \alpha_M (q_t - \sigma_{v0}) \tag{5.25}$$

When I_c <2.2 (coarse grained soils):

$$\alpha_M = 0.0188\left[10^{(0.55 I_c + 1.68)}\right] \tag{5.26}$$

I_c has been defined in Eq. (5.11).

5.5.4 Correlation with Small Strain Shear Modulus of Cohesionless Soils

Small strain shear modulus, G_0, is considered valid for very small strain levels up to 0.001 %. It is generally recognized that the most appropriate way of assessing G_0 is by measuring the shear wave velocity, V_s.G_0 could then be calculated as follows:

$$G_0 = \rho V_s^2 \tag{5.27}$$

where ρ = bulk density

However, unless project specific detailed investigations are carried out, it is generally the practice to use penetration tests to assess G_0 using empirical correlations.

Lunne et al. (1997) reports the correlation proposed by Rix and Stokes (1992) for uncemented quartz sands, shown in Eq. (5.28) and Fig. 5.8, which is based on calibration chamber test results.

$$\left(\frac{G_0}{q_c}\right)_{ave} = 1634\left(\frac{q_c}{\sqrt{\sigma'_{vo}}}\right)^{-0.75} \tag{5.28}$$

where G_0,q_c and σ_{vo}' are given in kPa in the range equal to Average ±Average/2.

Schnaid (2009) presented a theoretical relationship for G_0 against q_c of unaged cemented soils as shown in Fig. 5.9. Together with the theoretically derived database, Schnaid (2009) has shown empirically established upper and lower bounds on the same plot.

Robertson and Cabal (2014) state that available empirical correlations to interpret in situ tests apply to unaged, uncemented silica sands. To use such equations for other sands could lead to erroneous assessments in parameters, in the current case, G_0. Therefore they proposed the following lower and upper boundaries to characterize uncemented, unaged sands based on Eslaamizaad and Robertson (1997):

$$G_0 = b\left(q_t \sigma'_{v0}\, p_a\right)^{0.3} \tag{5.29}$$

where 'b' is a constant of 280 for upper bound and 110 for lower bound.

It is a narrow range and if the results fall outside the validity of the basic assumption should be checked, for example higher values indicating possible cementation or ageing.

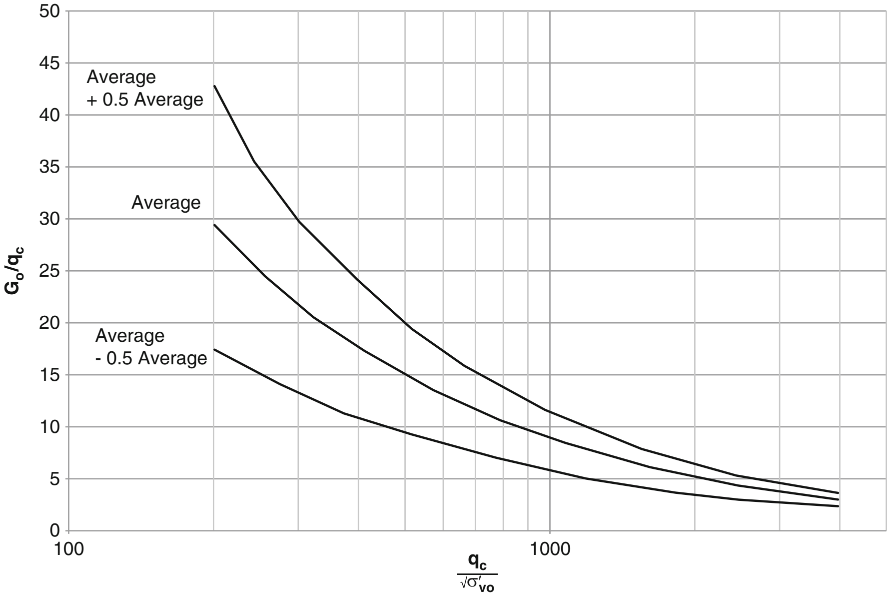


Fig. 5.8 G_o/q_c (Adapted from Rix and Stokes 1992)

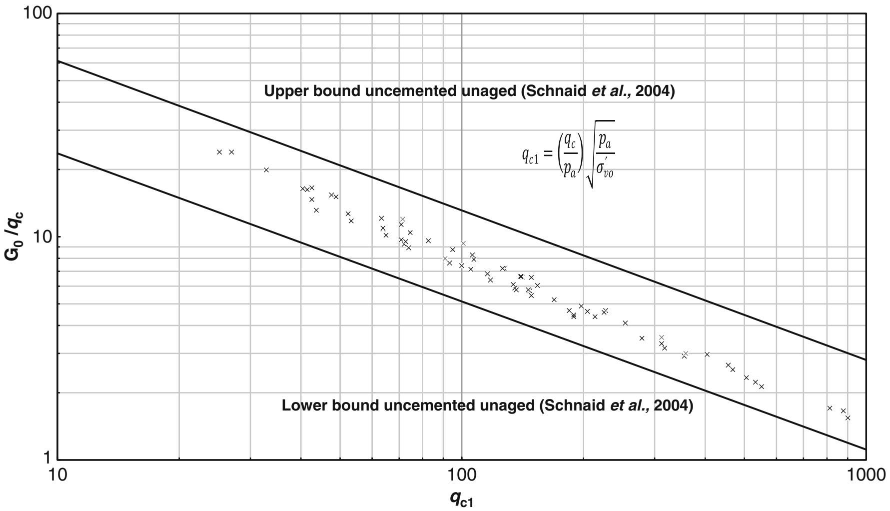


Fig. 5.9 Theoretical correlation between G_o/q_c and q_{c1} (Adapted from Schnaid 2009)

5.6 Correlations for Cohesive Soils

5.6.1 Correlation with Undrained Shear Strength of Cohesive Soils

Undrained shear strength of a clayey soil is one of the most derived parameters from a CPT test, apart from classification of the soil profile. The CPT test cannot directly measure the shear strength and only could be derived by empirical correlations associated with other tests, either in situ or laboratory. It is universally known and highlighted elsewhere in this book that the measured undrained shear strength is not unique and that it depends on many factors including rate of strain, loading arrangement, anisotropy, stress history etc. Therefore, when the CPT test result is calibrated against a particular test whether it is the vane test, direct shear or triaxial compression, the resulting values would be related to that particular test.

The commonly adopted test to calibrate against CPT is the vane shear test although other tests such as triaxial tests are also used.

Extensive research and studies have been conducted over the years and various theories postulated. However, the common, standard expression adopted by practitioners to derive c_u is based on Terzaghi's bearing capacity equation and can be written as follows:

$$c_u = \frac{(q_c - \sigma_{vo})}{N_c} \tag{5.30}$$

where N_c is called the cone factor.

With the emergence of CPTu and the measurement of pore pressures, as previously discussed, the tip resistance q_c measured by CPTu needs to be corrected to yield q_t (see Eq. (5.3)), and Eq. (5.30) could be re-written with a different cone factor N_{kt}.

$$c_u = \frac{(q_t - \sigma_{vo})}{N_{kt}} \tag{5.31}$$

Many studies have predicted N_{kt} values in the region of 10–20, sometimes even outside this range, and therefore a universally accepted unique value is not possible. It is strongly recommended that additional tests such as vane shear, direct shear, triaxial tests be conducted at test locations adjacent to cone penetrometer tests and calibrate to establish site specific N_{kt} values. Where such luxury is not available and there is no previous experience in the particular soil deposit, practitioners tend to adopt an N_{kt} value in the range of 14–16. However, it is recommended that sensitivity analysis be carried out especially where it is critical to the design.

Several researchers have attempted to identify correlations between the cone factor and the plasticity of the material. Aas et al. (1986) concluded that N_{kt} increases with plasticity although there was significant scatter in the results. Discussion by Powell and Quarterman (1988) and recent research by Kim et al. (2010) appear to confirm the conclusions of Aas et al. Robertson and Cabal (2014) also state that N_{kt} tends to increase with plasticity and decrease with sensitivity. However, others have found either no correlation or, in fact, N_{kt} decreasing with increasing plasticity index (La Rochelle et al. 1988; Hong et al. 2010; Remai 2013). Kulhawy and Mayne (1990) mention the work by Battaglio et al. (1973) who concluded that a trend exists for N_k for uncorrected vane shear test data in terms of plasticity index (*PI*). However, later reanalysis after correcting the vane hear test data indicated that N_k is not influenced by *PI*. Until further research confirms a correlation it is advised not to make any judgment unless site specific data is available.

It is suspected that the cone resistance is subjected to errors especially in softer clays. Therefore, researchers have proposed an alternative to derive c_u from excess pore pressure measurements. The corresponding equation could be written as follows:

$$c_u = \frac{\Delta u}{N_{\Delta u}} \tag{5.32}$$

where

$\Delta u = u_2 - u_0 =$ excess pore pressure measured at u_2 position and
$N_{\Delta u} =$ Pore pressure cone factor

Equations (5.7), (5.31) and (5.32) lead to the following relationship between $N_{\Delta u}$ and N_{kt}:

$$N_{\Delta u} = B_q N_{kt} \tag{5.33}$$

Robertson and Cabal (2014) indicate that $N_{\Delta u}$ varies between 4 and 10. Findings of other researchers found values of similar order, perhaps slightly narrower range (La Rochelle et al. 1988).

5.6.2 Correlation with Sensitivity of Cohesive Soils

Sensitivity can be defined as follows:

$$Sensitivity\,(S_t) = \frac{peak\,undisturbed\,shear\,strength}{remoulded\,shear\,strength}$$

One would deduce that sensitivity is somewhat related to the skin friction measured by the cone penetrometer. Schmertmann (1978) proposal was based on this and could be simply written as follows:

$$S_t = \frac{N_S}{R_f} \tag{5.34}$$

where N_s is a constant and R_f is the friction ratio.

Although Schmertmann (1978) suggested values for N_s it was based on a mechanical cone and therefore not applicable to electric cones including piezocones. While several proposals have been put forward, Lunne et al. (1997) suggestion of an average value of 5 with 6–9 likely range appears practical and reasonable.

5.6.3 Correlation with Over Consolidation Ratio of Cohesive Soils

Over consolidation ratio is an important parameter for cohesive soil deposits and is expressed as the ratio of the maximum past effective stress a soil element had ever been subjected to and the current effective vertical stress.

$$Over\,consolidation\,ratio\,(OCR) = \frac{maximum\,past\,effective\,pressure}{current\,effective\,vertical\,stress} = \frac{\sigma'_p}{\sigma'_v} \tag{5.35}$$

There are several methods to derive OCR from cone penetrometer data and some of the common or simple methods are given below.

Method 1 (Based on Mayne 2007).

- Estimate the overburden pressure σ_v
- Calculate σ'_p using Eq. 5.36

$$\sigma'_p = k(q_t - \sigma_v) \tag{5.36}$$

where k is a preconsolidation cone factor. An average of 0.33 is proposed with an expected range of 0.2–0.5

- Assess σ'_v and calculate OCR

Mayne (2007) points out that this is a first order estimate of the OCR for intact clays; it underestimates values for fissured clays. A similar order of factor k is recommended by Demers and Leroueil (2002)

Method 2

- Estimate σ'_v
- Estimate c_u from CPT or CPTu as discussed previously in this Section
- Calculate c_u/σ'_v (i.e. $(c_u/\sigma'_v)_{OC}$)
- Adopt $(c_u/\sigma'_p)_{NC}$ (say 0.22 as per Eq. (8.16)

Table 5.7 Constrained modulus coefficient for cohesive soils (Adopted from Sanglerat 1972; after Frank and Magnan 1995)

Soil type	q_c (MPa)	α
Low Plasticity Clay (CL)	$q_c < 0.7$	$3 < \alpha < 8$
	$0.7 < q_c < 2$	$1 < \alpha < 5$
	$q_c > 2$	$1 < \alpha < 2.5$
Silt, Low Liquid Limit (ML)	$q_c < 2$	$3 < \alpha < 6$
	$q_c > 2$	$1 < \alpha < 2$
High Plasticity Clay (CH)/ Silt, High Liquid Limit (MH)	$q_c < 2$	$2 < \alpha < 6$
	$q_c > 2$	$1 < \alpha < 2$
Organic Silt (OL)	$q_c < 1.2$	$2 < \alpha < 8$
Organic Clay (OH)/Peat (Pt) $50 < w < 100$ $100 < w < 200$ $w > 300$	$q_c < 0.7$	 $1.5 < \alpha < 4$ $1 < \alpha < 1.5$ $\alpha < 0.4$

Note – w = natural moisture content (%)

- Calculate *OCR* using Eq. (5.37)

$$\left(\frac{c_u}{\sigma_v'}\right)_{OC} = \left(\frac{c_u}{\sigma_v'}\right)_{NC} OCR^m \tag{5.37}$$

where m could be assumed to be 0.8 (see Sect. 8.6).

Method 3 (Based on Mayne and Kemper 1988)

$$OCR = 0.37\left(\frac{q_c - \sigma_o}{\sigma_o'}\right)^{1.01} \tag{5.38}$$

5.6.4 Correlation with Constrained Modulus of Cohesive Soils

Constrained modulus of soft to firm clays is commonly assessed using the consolidation test:

$$M = 1/m_v \tag{5.39}$$

where m_v is the coefficient of volume compressibility.

Table 5.7 and Eq. (5.40) from Frank and Magnan (1995), who cites the work of Sanglerat (1972), could be used to obtaine the constrained modulus. Equation (5.40) is similar in nature to Eq. (5.23) for sands.

Table 5.8 Constrained modulus factor for clays

Author	α_M
Meigh (1987)	2–8
Kulhawy and Mayne (1990)	8.25
Mayne (2001)	8
Robertson (2009)	Q_t for $Q_t < 14$ when $I_c > 2.2$
	14 for $Q_t > 14$ when $I_c > 2.2$
	$0.03\,[10^{(0.55 I_c + 1.68)}]$ when $I_c < 2.2$

Table 5.9 Coefficient of constrained modulus factor for NC and lightly OC clays and silts (After Meigh (1987))

Soil	Classification	α_M
Highly plastic clays and silts	CH, MH	2.5–7.5
Clays of Intermediate Plasticty $q_c < 0.7 MN/m^2$ $q_c > 0.7 MN/m^2$	CI, CL	 3.7–10 2.5–6.3
Silts of Intermediate or low Plasticity	MI, ML	3.5–7.5
Organis silts	OL	2.5–10
Peat and organic clay $50 < w < 100$ % $100 < w < 200$ % $w > 200$ %	Pt, OH	 1.9–5 1.25–1.9 0.5–1.25

$$M = \alpha q_c \tag{5.40}$$

More recent studies indicate that M could be obtained from CPT profile using Eq. (5.41) which is of a similar form to Eq. (5.25) for sands.

$$M = \alpha_M (q_t - \sigma_{vo}) \tag{5.41}$$

Some of the values for α_M reported in the literature are shown in Table 5.8.

A more detailed assessment for different materials was carried out by Sanglerat (1972). However, as Meigh (1987) points out the recommendations were based on a Dutch Cone penetrometer. Meigh (1987), based on limited field evidence, adopted a factor (up) of 1.25 to the values proposed by Sanglerat (1972) and the modified factors are shown in Table 5.9.

5.6.5 *Correlation with Compressibility of Cohesive Soils*

Centre for Civil Engineering Research and Codes (CUR) (1996) suggests the following formula to calculate the compression ratio, CR:

Table 5.10 Coefficient β in Eq.(5.42) (After CUR 1996)

Soil type	Coefficient β
Sandy clay	0.2–0.4
Pure clay	0.4–0.8
peat	0.8–1.6

$$\frac{1}{CR} = \frac{q_c}{2.3\beta\sigma_v'} \tag{5.42}$$

The coefficient β for different soil types of cohesive materials is presented in Table 5.10.

5.6.6 Correlation with Friction Angle of Cohesive Soils

For soft to firm clays where c' can be assumed to be zero, Eq. (5.43) could be used to find ϕ' where it is related to B_q (Mayne 2014). The assumption of c′=o preclude the use of this equation to assess the friction angle of OC soils. Mayne (2014) states that the equation is only applicable if $20° < \phi' < 45°$ and $0.1 < B_q < 1.0$.

$$\phi' \,(degrees) = 29.5 B_q^{0.121} \left[0.256 + 0.336 B_q + \log Q_t\right] \tag{5.43}$$

Where $Q_t = \frac{q_t - \sigma_{vo}}{\sigma_{vo}'}$ as previously defined.

5.6.7 Correlation with Small Strain Shear Modulus of Cohesive Soils

As previously discussed, the small strain shear modulus is related to the shear wave velocity, v_s. Mayne and Rix (1995) presented the following equation to obtained v_s from q_c values of intact and fissured clays.

$$v_s = 1.75\,(q_c)^{0.627} \tag{5.44}$$

where v_s is in units of m/s and q_c in kPa.

By considering as a bearing capacity problem, Mayne and Rix (1993) stated that it is more appropriate to utilize a net cone resistance, such as $(q_c - \sigma_{vo})$, or more correctly $(q_t - \sigma_{vo})$. The data for both fissured and intact clays were re-examined but no statistical improvement could be detected. Therefore only the intact clay results were re-examined and the following correlation resulted:

Table 5.11 CPT – V_s Correlation equations (After Wair et al. 2012)

Soil Type	Study	Geological Age	V_s(m/s)
All soils	Hegazy and Mayne (1995)	Quaternary	$\{10.1\log(q_c) - 11.4\}^{1.67}\ \left(100^{f_s}/_{q_c}\right)^{0.3}$
	Mayne (2006)	Quaternary	$118.8\log(f_s) + 18.5$
	Piratheepan (2002)	Holocene	$32.3\ q_c^{0.089}\ f_s^{0.121}\ D^{0.215}$
	Andrus et al. (2007)	Holocene & Pleistocene	$2.62\ q_t^{0.395}\ I_c^{0.912}\ D^{0.124}\ SF^a$
	Robertson (2009)	Quaternary	$\left\{\left(10^{(0.55I_c+1.68)}\right)(q_t - \sigma_v)/p_a\right\}^{0.5}$
Sand	Sykora and Stokoe (1983)	–	$134.1 + 0.0052\ q_c$
	Baldi et al. (1989)	Holocene	$17.48\ q_c^{0.33}\sigma_v'^{\ 0.27}$
	Hegazy and Mayne (1995)	Quaternary	$13.18\ q_c^{0.192}\sigma_v'^{\ 0.179}$
	Hegazy and Mayne (1995)	Quaternary	$12.02\ q_c^{0.319}\ f_s^{-0.0466}$
	Piratheepan (2002)	Holocene	$25.3\ q_c^{0.163}\ f_s^{0.029}\ D^{0.155}$
Clay	Hegazy and Mayne (1995)	Quaternary	$14.13\ q_c^{0.359}\ e_0^{-0.473}$
	Hegazy and Mayne (1995)	Quaternary	$3.18\ q_c^{0.549} f_s^{0.025}$
	Mayne and Rix (1995)	Quaternary	$9.44\ q_c^{0.435}\ e_0^{-0.532}$
	Mayne and Rix (1995)	Quaternary	$1.75\ q_c^{0.627}$
	Piratheepan (2002)	Quaternary	$11.9\ q_c^{0.269}\ f_s^{0.108}\ D^{0.127}$

p_a =100kPa ; $^aSF = 0.92$ for Holocene and 1.12 for Pleistocene
Stress unit in kPa and depth (D) in meters

$$v_s = 9.44\,(q_c)^{0.435}(e_0)^{-0.532} \tag{5.45}$$

where v_s is in m/s and q_c in kPa.

Mayne and Rix (1993) developed the following equation for clays where G_{max} is the initial tangent shear modulus (i.e. same as G_0)

$$G_{max} = 99.5\ p_a^{\ 0.305}\,(q_c)^{\frac{0.695}{e_0^{\ 1.130}}} \tag{5.46}$$

where p_a , G_{max} and q_c of same units and e_0 is the initial void ratio.

Various other authors have provided correlations for v_s and q_c. Wair et al. (2012) summarized v_s prediction equations as shown in Table 5.11. For consistency, Wair et al. (2012) have modified the equations to use consistent units of kPa for q_c, f_s and σ'_v with depth in metres.

5.7 Correlation with Unit Weight

There have been several studies linking the unit weight to CPT measurements. There have also been correlations for the unit weight involving the shear wave velocity. Two of the recent correlations for unit weight derivation based on measurements of CPT are by Robertson and Cabal (2010) and Mayne et al. (2010) who have used the sleeve friction in addition to the tip resistance to propose useful relationships. Robertson and Cabal (2010) proposal is shown in Fig. 5.10 and in Eq. (5.56) while Mayne et al. (2010) derivation is shown in Eq. (5.47).

$$\frac{\gamma}{\gamma_w} = 0.27\left[logR_f\right] + 0.36\left[log\left({}^{q_t}/_{p_a}\right)\right] + 1.236 \tag{5.47}$$

$$\gamma_t = 1.95\gamma_w\left(\frac{\sigma'_{vo}}{p_a}\right)^{0.06}\left(\frac{f_s}{p_a}\right)^{0.06} \tag{5.48}$$

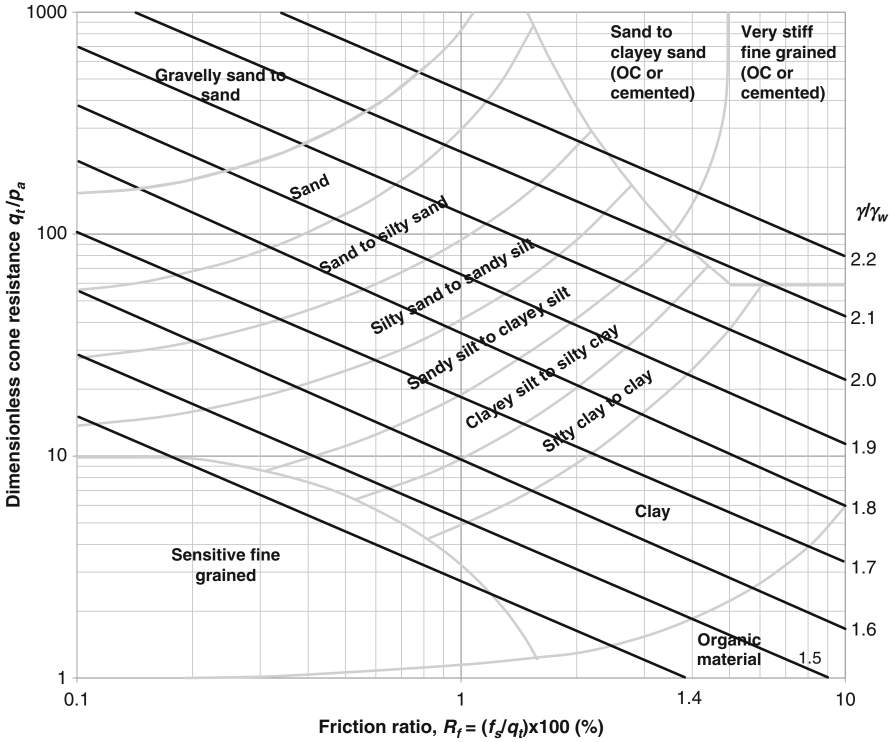


Fig. 5.10 Normalised unit weight and friction ratio (Adapted from Robertson and Cabal 2010)

Table 5.12 Permeability from CPT results

Soil Behaviour Type	Soil Permeability (m/s)
Sensitive fine grained	3×10^{-9} to 3×10^{-8}
Organic soils-peats	1×10^{-8} to 1×10^{-6}
Clays-clay to silty clay	1×10^{-10} to 1×10^{-7}
Silt mixtures clayey silt to silty clay	3×10^{-9} to 1×10^{-7}
Sand mixtures; silty sand to sandy silt	1×10^{-7} to 1×10^{-5}
Sands; clean sands to silty sands	1×10^{-5} to 1×10^{-3}
Gravelly sand to sand	1×10^{-3} to 1
[a]Very stiff sand to clayey sand	1×10^{-8} to 1×10^{-6}
[a]Very stiff fine grained	1×10^{-9} to 1×10^{-7}

[a]Over consolidated or cemented

5.8 Correlation with Permeability

Robertson (1990) provided a correlation between the Soil Behaviour Type (see Sect. 5.4) and the permeability as shown in Table 5.12. Robertson and Cabal (2010) state that the average permeability (k) shown in Table 5.12 can be represented by Eq. (5.49) and Eq. (5.50) using SBT index, I_c, defined by Eq. (5.11).

$$When \qquad 1.0 < I_c \leq 3.27 \qquad k = 10^{(0.952-3.04\, I_c)}\ m/s \tag{5.49}$$

$$When \qquad 3.27 < I_c \leq 4.0 \qquad k = 10^{(-4.52-1.37\, I_c)}\ m/s \tag{5.50}$$

5.9 Correlation with SPT *N*

Standard penetration test (SPT) is probably the most used in situ device worldwide to assess the soil characteristics in situ. Some designers prefer to convert CPT results to equivalent SPT *N* values if the design methods are based on SPT *N* value, and vice versa.

As the CPT is a continuous test it provides more details than SPT test results because of the nature of the test being continuous and therefore detects local changes in the profile. This is most evident in soft to firm soils where SPT *N* values are generally found to be zero or a very low value. Such results do not provide a great deal of information to the designer except perhaps that the clay is very soft or soft. Conversely, the CPT provides significantly more details which could be used to interpret standard parameters. This is evident in the example shown in Fig. 5.11a which shows a cluster of CPT results converted to c_u values. In the same Figure, available vane shear test results, some of which used to calibrate CPT results, are also plotted. Figure 5.11b shows the available SPT N values from boreholes located in the same area. It is evident that the information available for the weak layers from the SPT results is significantly less useful to the designer than CPT plots. For stiffer soils and sandy soils, SPT N values provide much more useful information and the reason its use worldwide in routine foundation designs.

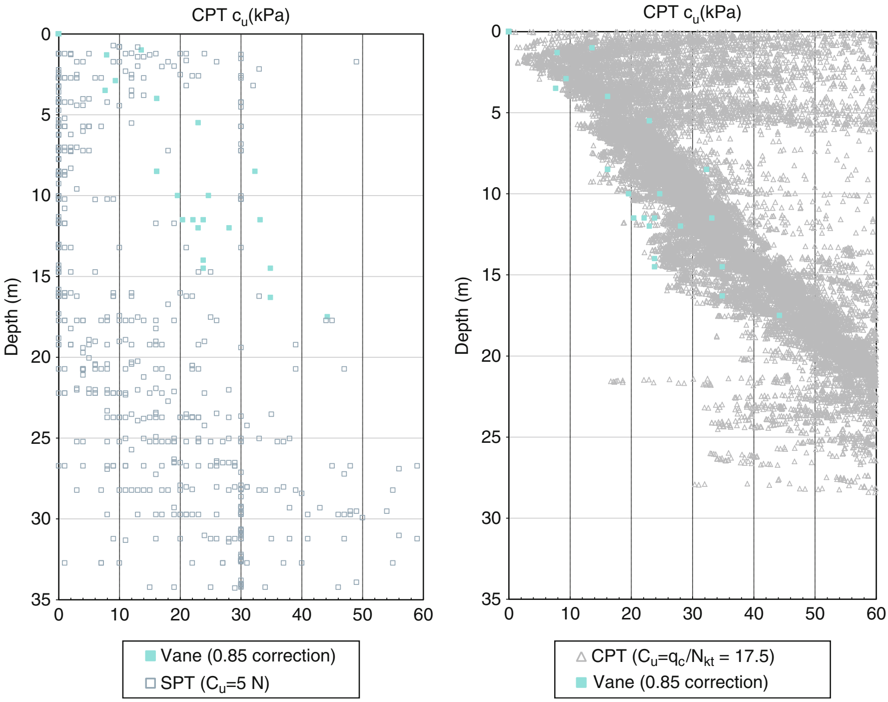


Fig. 5.11 Comparison of SPT and CPT profiles in a weak clay deposit

Table 5.13 Ratios of q_c/N (After Sanglerat 1972; Schmertmann 1970, 1978)

Soil	q_c (kPa)/N_{60}
Silts, sandy silts, slightly cohesive silt-sand mix	200[a] (200–400)[b]
Clean fine to medium sands and slightly silty sands	300–400[a] (300–500)[b]
Coarse sands and sands with little gravel	500–600[a] (400–500)[b]
Sandy gravel and gravel	800–1000[a] (600–800)[b]

[a]Values proposed by Sanglerat (1972) and reported in Peck et al. (1974)
[b]Values suggested by Schmertmann (1970, 1978), reported by Holtz (1991) in parentheses

When results of both types of tests are not available CPT-SPT relationships become very useful to the designer. Various authors have provided CPT-SPT relationships mostly based on experienced gained from in situ testing and calibrating against each other.

Sivakugan and Das (2010) summarized the work of Sanglerat (1972) and Schmertmann (1970, 1978) on the ratio of q_c/N for different soils, as shown in Table 5.13. The results indicate that the ratio is low, 200–400 (kPa), for fine grained materials, increasing to 800–1000 (kPa) for gravels.

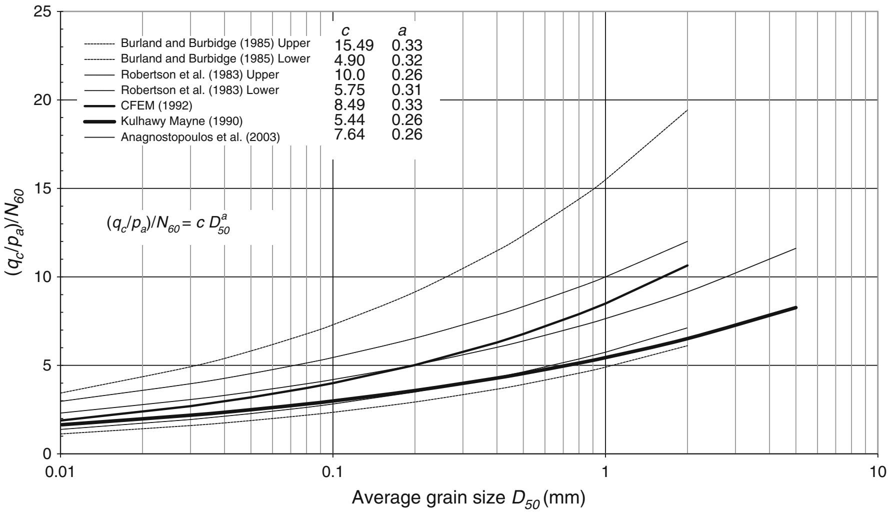


Fig. 5.12 q_c/N_{60} variation with grain size (Adapted from Robertson and Campanella 1983)

As Robertson and Campanella (1983) investigated they found that q_c/N ratio values published had a very wide scatter. However they found that the results could be rationalized somewhat when the ratio is pitted against the mean grain size, D50. (see Fig. 5.12).

The importance of the grain size in penetration test results is evident from Fig. 5.12, the plot of the variation of q_c/N with the median grain size D_{50}, and the upper and lower bounds, from Robertson et al. (1983). In the same Figure, data from Burland and Burbidge (1985), Canadian Foundation Engineering Manual (Canadian Geotechnical Society 1992), Kulhawy and Mayne (1990) and Anagnostopoulos et al. (2003) are also shown, some as lower and upper bounds or average values. The curves shown in Fig. 5.12 take the form of:

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = c\,D_{50}{}^{a} \tag{5.51}$$

where the values of '*a*' and '*c*' are shown in Fig. 5.12.

Some of these relationships are based on a significant data base and/or relevant to a particular locality and this is one of the reasons for the differences in the coefficients '*a*' and '*c*'. For example, Kulhawy and Mayne (1990) equation has the coefficients $c = 5.44$ and $a = 0.26$:

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = 5.44\,D_{50}^{0.26} \tag{5.52}$$

However, the proposed relationship by Anagnostopoulos et al. (2003) for Greek soils has $c = 7.64$ although a remains the same:

$$\frac{\left(\frac{q_c}{p_a}\right)}{N_{60}} = 7.64 D_{50}^{0.26} \tag{5.53}$$

To use the above relationships the median grain size (D_{50}) is required. If test results are not available, D_{50} needs to be estimated. Jefferies and Davies (1993) proposed a method for the estimation of N_{60} values directly from CPTu results without resorting to soil sampling and laboratory testing. They proposed the use of soil behavior type (SBT) index (see Sect. 5.4) to correlate CPT and SPT. Lunne et al. (1997) applied this technique to the SBT Index, I_c, defined in Eq. (5.11) to give the following relationship.

$$\frac{q_c/_{p_a}}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right) \tag{5.54}$$

where I_c = Soil Behaviour Type in Eq.(5.11)

Jefferies and Davies (1993) suggest that the CPTu has a fivefold improved precision compared to the SPT. Robertson and Cabal (2014) agree with Jeffries and Davies (1993) that the most reliable way to obtain SPT N values is to perform a CPT and convert the CPT to an equivalent SPT.

5.10 Correlation with Bearing Capacity

5.10.1 Shallow Foundations

De Court (1995) proposed the following correlation with tip resistance, q_c, from electric CPT to calculate the ultimate bearing capacity of a shallow foundation for sand:

$$Ultimate\,bearing\,capacity = q_{ult} = \frac{q_c}{4} \tag{5.55}$$

Frank and Magnan (1995) citing the French experience state the bearing capacity of shallow foundations according to MELT (1993) is as follows:

$$q_{ult} = k_c q_c + q_o \tag{5.56}$$

where k_c is the bearing factor given in Table 5.14.

Table 5.14 Bearing capacity factor k_c (CPT) for shallow foundations (After MELT 1993)

Soil type		Expression for k_c
Clay/Silt	Soft to Hard	$0.32\left[1+0.35\left(0.6+0.4\frac{B}{L}\right)\frac{D}{B}\right]$
Sand/Gravel	Loose	$0.14\left[1+0.35\left(0.6+0.4\frac{B}{L}\right)\frac{D}{B}\right]$
	Medium	$0.11\left[1+0.50\left(0.6+0.4\frac{B}{L}\right)\frac{D}{B}\right]$
	Dense	$0.08\left[1+0.85\left(0.6+0.4\frac{B}{L}\right)\frac{D}{B}\right]$
Chalk	Weathered	$0.17\left[1+0.27\left(0.6+0.4\frac{B}{L}\right)\frac{D}{B}\right]$

B = width; L = length; D = embedment

Table 5.15 Base bearing capacity factor k_c (CPT) for deep foundations (After MELT 1993)

Soil type		q_c (MPa)	k_c ND	k_c D
Clay/Silt	soft	<3	0.40	0.55
	stiff	3–6		
	hard (clay)	>6		
Sand/Gravel	loose	<5	0.15	0.50
	medium	8–15		
	dense	>20		
Chalk	soft	<5	0.20	0.30
	weathered	>5	0.30	0.45

ND = non displacement pile, D= displacement pile

5.10.2 Deep Foundations

Frank and Magnan (1995) in their national report for France for the CPT95 conference state that the French practice does not use the sleeve friction f_s to calculate bearing capacity but only the cone resistance q_c. The ultimate base resistance is given by:

$$q_b = k_c\, q_c \tag{5.57}$$

where k_c is the base bearing capacity factor.

The values of k_c recommended by the design code for foundations of the French road administration (MELT 1993) are given in Table 5.15. The unit skin friction q_s is obtained from using the following equation.

$$q_s = \text{minimum value of } \{q_c/\beta;\ q_{smax}\} \tag{5.58}$$

where β and q_{smax} values recommeneded for different soil types and pile types are given in Table 5.16. Frank and Magnan (1995) state that CPT pile design rules used in France are based on a large database of full scale pile loading tests and therefore probably one of the best design rules used.

Table 5.16 Unit limit skin friction from CPT (After MELT 1993)

Pile type		Clay and silt					Sand and gravel			Chalk	
		Soft	Stiff		Hard		Loose	Medium	Dense	Soft	Weathered
Drilled	β	–	–	75[a]	–	–	200	200	200	125	80
	q_{smax} (kPa)	15	40	80[a]	40	80[a]	–	–	120	40	120
Drilled removed casing	β	–	100	100[b]	–	100[b]	250	250	300	125	100
	q_{smax} (kPa)	15	40	60[b]	40	80[b]	–	40	120	40	80
Steel driven closed-ended	β	–	120		150		300	300	300	[c]	
	q_{smax} (kPa)	15	40		80		–	–	120		
Driven concrete	β	–	75		–		150	150	150	[c]	
	q_{smax} (kPa)	15	80		80		–	–	120		

[a]Trimmed and grooved at the end of drilling
[b]dry excavation, no rotation of casing
[c]in chalk, q_s can be very low for some types of piles; a specific study is needed

5.11 Liquefaction Assessment

As discussed in Sect. 4.11 in Chap. 4, soil liquefaction is a serious phenomenon which could create major destruction to life and property during earthquakes. The use of in situ tests to characterize soil liquefaction is quite popular and it is widely accepted that they provide better solutions than laboratory tests because of inherent difficulties associated with collecting undisturbed samples from sandy soils. Historically, SPT test results were used to assess liquefaction potential, however, with the advent and rapid advancement of the CPT, the latter has provided an additional tool that could be used for the same purpose. Table 5.17 lists the advantages and disadvantages of the two methods.

CPT is now accepted as a primary tool for liquefaction assessment because of, as Robertson and Campanella (1995) point out, simplicity, repeatability and accuracy. CPT also provides a continuous record in addition to being quicker and less costly. It is quite common to use both SPTs and CPTs especially when the project is large and/or critical.

Liquefaction potential is generally assessed based on a factor of safety related to the soil resistance. The factor of safety (FS_{li}) is defined as the ratio of cyclic shear strength of the soil (i.e., cyclic resistance ratio, *CRR*) and the cyclic stress developed by the design earthquake (i.e., cyclic stress ratio, *CSR*):

$$FS_{li} = CRR/CSR \tag{5.59}$$

The use of appropriate FS_{li} depends on the site, information available, assessment tools and design assumptions made. If design assumptions are conservative and good quality data are available and the designer adopts a conservative approach a FS_{li} of 1 could be adopted if valid computational method is used. Even then, a higher value may be warranted if the consequences of failure could have a significant effect on the environment and health and safety.

Table 5.17 Comparison of SPT and CPT for assessment of liquefaction potential (After Youd et al. 2001)

Feature	SPT	CPT
Number of test measurements at liquefaction sites	Abundant	Abundant
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain
Quality control and repeatability	Poor to good	Very good
Detection variability of soil deposits	Good	Very good
Soil types in which test is recommended	Non-gravel	Non-gravel
Test provides sample of soil	Yes	No
Test measures index or engineering property	Index	Index

CRR and *CSR* values are often adjusted to an equivalent shear stress induced by an earthquake magnitude $M = 7.5$ and commonly referred to as $CRR_{7.5}$ and $CSR_{7.5}$ and therefore Eq. (5.59) could be written as follows:

$$FS_{li} = {}^{CRR_{7.5}}\big/_{CSR_{7.5}} \tag{5.60}$$

The methodology to assess liquefaction potential or FS_{li} could be summarized as follows (Idriss and Boulanger 2006):

1. Calculate *CSR* for the design earthquake, M (CSR_M) – Sect. 5.11.1)
2. Calculate Magnitude Scaling Factor (*MSF*) – Sect. 5.11.1.1
3. Convert CSR_M to a standard earthquake magnitude of 7.5 (i.e. $CSR_{7.5}$) – Sect. 5.11.1
4. Normalise resistance and correct for overburden stress – Sect. 5.11.2

   (a) CPT– Sect. 5.11.2.1
   (b) SPT – Sect. 5.11.2.2

5. Correct for Fines Content, *FC* – Sect. 5.11.2.3
6. Calculate *CRR* for a magnitude of 7.5 and 1 atmosphere, i.e. $CRR_{7.5,100}$ – Sect. 5.11.3
7. Calculate *CRR* – Sect. 5.11.3 (after assessing the overburden factor, K_σ – Sect. 5.11.3.1)
8. Calculate FS_{li} – Sect. 5.11 introduction

5.11.1 Cyclic Stress Ratio

Cyclic Stress Ratio (*CSR*) for a design earthquake magnitude *M* could be calculated by the following (Seed and Idriss 1971):

$$CSR_M = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d \tag{5.61}$$

where

a_{max} maximum horizontal acceleration at the ground surface
g acceleration due to gravity
σ_{vo} total vertical overburden stress
σ'_{vo} effective overburden stress
r_d stress reduction factor

Idriss (1999) proposed the following expressions to compute the stress reduction factor, r_d:

$$r_d = e^{\left(\alpha_{(z)} + \beta_{(z)} M\right)} \tag{5.62a}$$

$$\alpha_{(z)} = -1.012 - 1.126\, Sin\left(\frac{z}{11.73} + 5.133\right) \tag{5.62b}$$

$$\beta_{(z)} = 0.106 - 0.118\, Sin\left(\frac{z}{11.28} + 5.142\right) \tag{5.62c}$$

z = depth below ground surface ($z \leq 34$m)

As Idriss and Boulanger (2008) state, although the above equations are mathematically applicable to a depth of $z \leq 34$ m, uncertainty in r_d increases with depth and therefore should only be applied to depths less than 20 m or so. For deeper sites specific site response analysis should be carried out.

Other correlations have been proposed and the following tri-linear function provides a good fit to r_d originally proposed by Seed and Idriss in 1971 (Youd et al. 2001):

$$r_d = 1.0 - 0.00765\, z \; if \; z \leq 9.15\, m \tag{5.63a}$$

$$r_d = 1.174 - 0.0267\, z \; if \; 9.15m < z \leq 23\, m \tag{5.63b}$$

Magnitude Scaling Factor (*MSF*) is used to convert the *CSR* calculated for the design earthquake to a common or reference earthquake magnitude, generally accepted to be of magnitude 7.5 using Eq. (5.64).

$$CSR_{7.5} = CSR_M / MSF \tag{5.64}$$

5.11.1.1 Magnitude Scaling Factor (*MSF*)

There are several expressions that could be used to calculate *MS*F. The participants at the NCEER Workshops in 1996/98 recommended the following expression as a lower bound for *MSF* (Youd et al. 2001):

$$MSF = {}^{10^{2.24}}\!/\!_{M^{2.56}} \tag{5.65}$$

The MSF relationship was re-evaluated by Idriss (1999) as reported by Idriss and Boulanger (2006) and presented below.

$$MSF = 6.9\, e^{\left(\frac{-M}{4}\right)} - 0.058 \leq 1.8 \tag{5.66}$$

5.11.2 Normalization of Resistance

The methodology described below is as described by Boulanger (2003) and, Idriss and Boulanger (2006) including fines correction proposed by Robertson and Wride (1998). To be consistent and simple, it is assumed that the unit for pressure/stress would be kPa.

5.11.2.1 Normalization of Resistance – CPT

CPT penetration resistance, q_c, is initially corrected for overburden stress effects using an equivalent effective vertical stress (σ'_{vo}) of one atmosphere (100 kPa) and an overburden correction factor, C_N, as part of the semi-empirical procedure (Boulanger 2003):

$$q_{cN} = \frac{q_c}{100} \tag{5.67}$$

$$q_{c1N} = C_N\, q_{cN} = C_N \frac{q_c}{100} \le 254 \tag{5.68}$$

$$C_N = \left(\frac{100}{\sigma'_{vo}}\right)^{\beta} \le 1.7 \tag{5.69}$$

$$\beta = 1,338 - 0.249\,(q_{c1N})^{0.264} \tag{5.70}$$

Solving for C_N requires an iterative process because of its dependence on q_{c1N}.

5.11.2.2 Normalization of Resistance – SPT

The equivalent equations for SPT are as follows (Boulanger 2003):

$$(N_1)_{60} = C_N\,(N)_{60} \tag{5.71}$$

$$C_N = \left(\frac{100}{\sigma'_{vc}}\right)^{\alpha} \le 1.7 \tag{5.72a}$$

$$\alpha = 0.784 - 0.0768\left\{(N_1)_{60}\right\}^{0.5} \tag{5.72b}$$

where σ'_{vc} is the operating effective vertical stress in kPa.

Solving for C_N requires an iterative process because of its dependence on $(N_1)_{60}$.

5.11.2.3 Correction for Fines Content

The above equations in Sects. 5.11.2.1 and 5.11.2.2 are based on the assumption that sands encountered are from clean sand deposits. It is generally accepted that correlations to obtain CRR values would be different if the sands have fines. Therefore a correction is made to adjust the SPT or CPT resistance to an equivalent clean sand value.

For the SPT test, the following relationship was proposed by Idriss and Boulanger 2004 for non-plastic sands:

$$\Delta(N_1)_{60} = \exp\left\{1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right\} \tag{5.73a}$$

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \tag{5.73b}$$

In the case of CPT, Robertson and Wride (1997) and Suzuki et al. (1997) proposed the use soil behavior type index which is a function of q_c and F_r to obtain CRR when the fines content is high. However, as Idriss and Boulanger (2008) state, the curve proposed by Robertson and Wride (1997) is unconservative with similar comments on the proposal by Suzuki et al. (1997) for high fine contents. Idriss and Boulanger (2008) suggest the modification of cone resistance values to account for non-plastic fines in a similar way to the SPT corrections discussed above using the following relationships:

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right)\exp\left\{1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right\} \tag{5.74a}$$

where FC = Fines content

$$(q_{c1N})_{cs} = q_{c1N} + \Delta q_{c1N} \tag{5.74b}$$

5.11.3 Computation of Cyclic Resistance Ratio (CRR)

The Cyclic Resistance Ratio, CRR, could be obtained from the CPT and SPT results using Eqs. (5.75) and (5.76) respectively (Idriss and Boulanger 2004):

$$CRR_{7.5,\,100} = \exp\left\{\left(\frac{(q_{c1N})_{cs}}{540}\right) + \left(\frac{(q_{c1N})_{cs}}{67}\right)^2 - \left(\frac{(q_{c1N})_{cs}}{80}\right)^3 + \left(\frac{(q_{c1N})_{cs}}{114}\right)^4 - 3\right\} \tag{5.75}$$

$$CRR_{7.5,\,100} = \exp\left\{\left(\frac{(N_1)_{60CS}}{14.1}\right) + \left(\frac{(N_1)_{60CS}}{126}\right)^2 - \left(\frac{(N_1)_{60CS}}{23.6}\right)^3 + \left(\frac{(N_1)_{60CS}}{25.4}\right)^4 - 2.8\right\} \tag{5.76}$$

where $CRR_{7.5,100}$ relates to an earthquake magnitude of 7.5 and an effective stress of 100kPa.

The value of $CRR_{7.5,100}$ needs to be adjusted to the overburden to obtain $CRR_{7.5}$. This is carried out by the use of an overburden correction factor, K_σ (Boulanger 2003):

$$CRR_{7.5} = CRR_{7.5,100}\,K_\sigma \tag{5.77}$$

Section 5.11.3.1 describes the procedure to obtain the relevant K_σ value.

5.11.3.1 Assessment of Overburden Factor

Boulanger (2003) proposed the following expression for K_σ:

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vc}}{100}\right) \leq 1.0 \tag{5.78}$$

where C_σ could be obtained from the following expressions for the CPT and SPT (Boulanger 2003):

$$C_\sigma = \frac{1}{37.3 - 8.27\,(q_{c1N})^{0.264}} \leq 0.3 \tag{5.79}$$

$$C_\sigma = \frac{1}{18.9 - 2.55\,\left((N_1)_{60}\right)^{0.5}} \leq 0.3 \tag{5.80}$$

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