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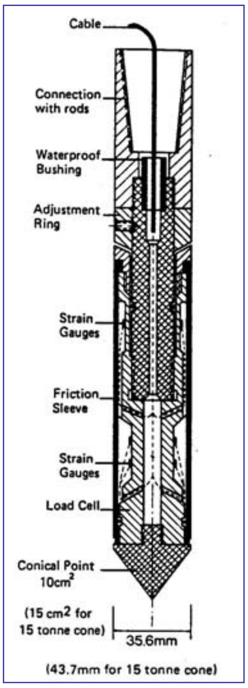
Cone penetration tests (CPT)

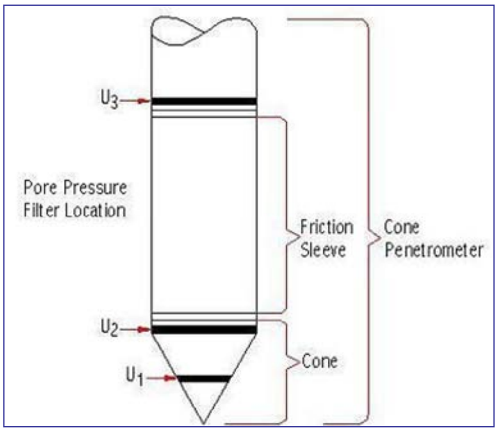
Piezocone tests (CPTU)

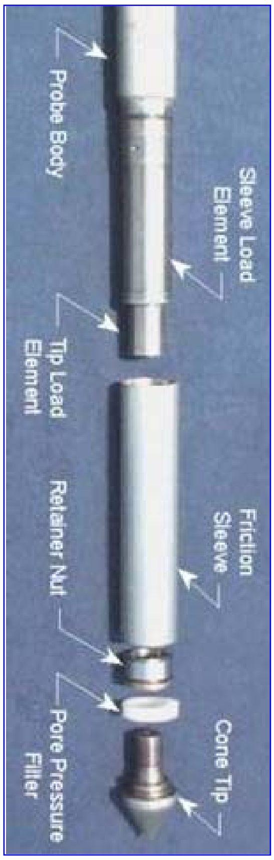
by David Nash

references:
Cone penetration testing in geotechnical practice. by Lunne, Robertson, Powell (pub 1997 Blackie)
In-situ testing in Geomechanics: the Main Tests by Fernando Schnaid (pub 2009 Taylor & Francis)

General arrangement of electric cone and piezocone







Electric Cone Penetration Tests, CPT & CPTU

The main features are:

- cone, with dia 35.7mm, area $10cm^2$. cone angle 60°
- friction sleeve, $150cm^2$ area
- load cells to measure the cone and friction sleeve forces, and pore pressure (in piezocone)
- electric cable up the centre of the rods to data acquisition system at the surface

The test is carried out by pushing continuously (except for delays to add rods) at 20mm/second, with the loads on the cone and friction sleeve being continuously recorded. With modern setup's the data is processed immediately and a plot of cone resistance q_c, sleeve resistance f_s and friction ratio R_f is produced in the field.

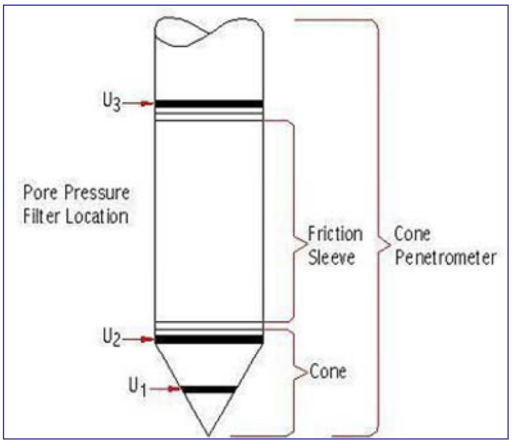
The mechanical cone is more labour intensive, and suffers from friction in the push-rods by measuring loads at the ground surface with a proving ring, provides a discontinuous record, and has a lower productivity than electric cones. It is rarely used in the UK.

Measurements are made of:

tip resistance $q_c = Q_c/A_c$

side friction $f_s = Q_s/A_s$

pore pressure u *(three possible locations)*



Calculations are made of:

corrected tip resistance $q_t = q_c + u_2.a$

where a is ratio of the area of the back of the cone affected by water pressure u to the area of the cone

net tip resistance $q_n = q_t - \sigma_{v0}$

normalised tip resistance $Q_t = (q_t - \sigma_{v0})/\sigma'_{v0}$

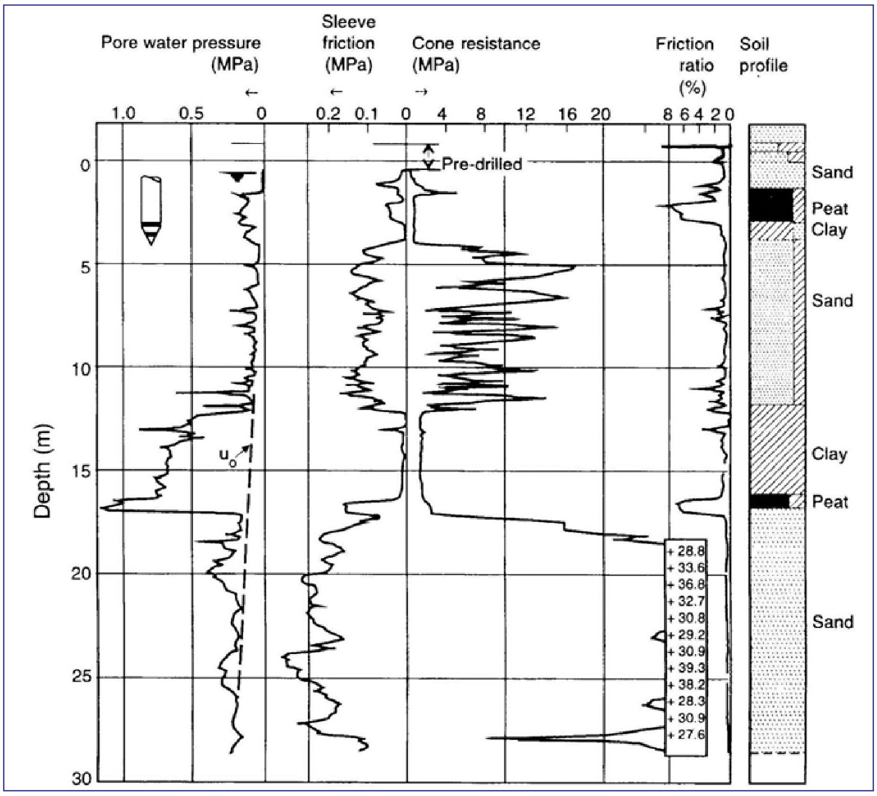
friction ratio $R_f = (f_s / (q_t - \sigma_{v0}) \times 100\%$

excess pore pressure ratio $B_q = (u_2 - u_0)/(q_t - \sigma_{v0})$

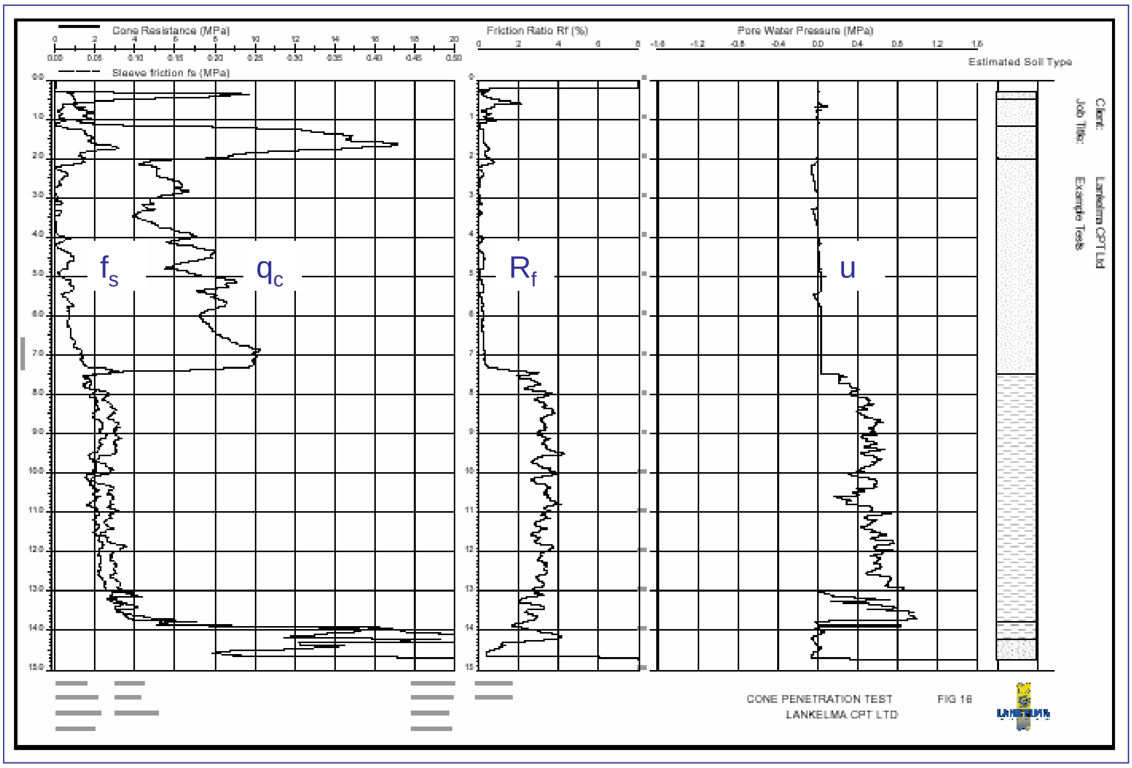
Output from the Tests:

The output from the CPT tests is usually a plot of q_c, f_s and R_f vs depth or q_c and R_f vs depth. For the CPTU, the output is usually a plot of q_c, f_s, R_f and u_{max} vs depth. u_{max} is the total pore pressure ie. static plus that generated by the action of inserting the cone.

Example of piezocone profile



Example of piezocone profile



Objectives of carrying out CPT or CPTu tests

Identify the likely stratigraphy from measured profiles of q_c f_s and u using derived parameters and classification charts.

For cohesive soils –

use correlations to estimate the OCR, undrained shear strength c_u

determine coefficient of consolidation c_h from dissipation tests.

For cohesionless soils –

use correlations to estimate the relative density, friction angle and stiffness;

assessment of liquefaction potential;

use results directly in pile design and settlement analysis.

With additional sensors the CPT can be used to measure seismic velocities, resistivity, and many other parameters.

Soil Classification

One of the most valuable attributes of CPT and CPTU testing is that the soil classification can be determined, allowing a continuous profile of the soil conditions to be obtained. This allows location of thin layers of different soil which might be overlooked in drilling and sampling/testing with thin walled sample tubes and SPT. The chart below is one of many that have been developed to allow identification of soils.

Robertson and Campanella's (1984) method for soil classification from CPT

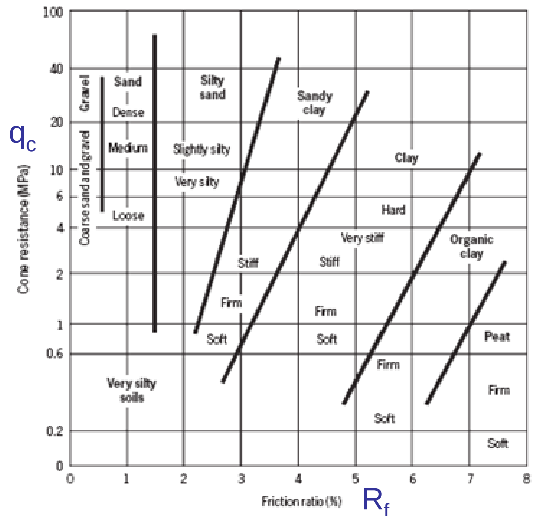
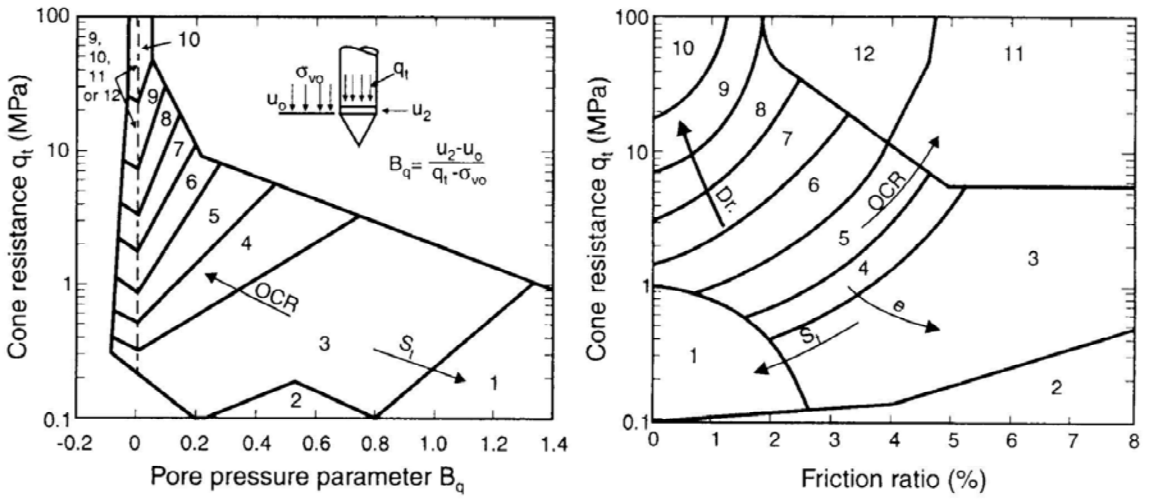


Figure 5 Chart for soil classification

Roberston and Campanella's 1986 chart



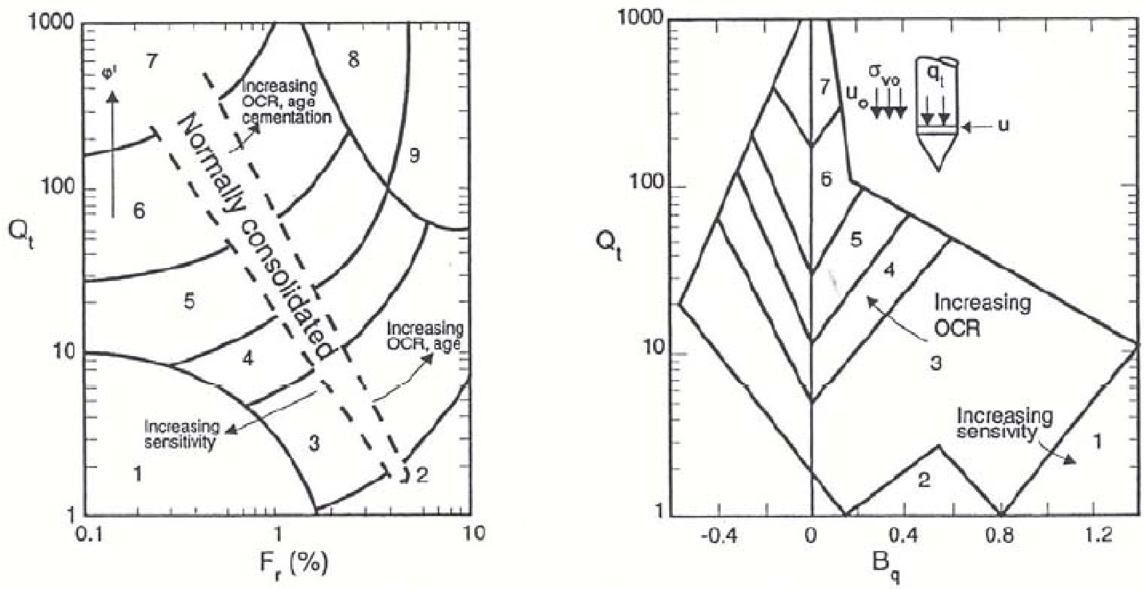
Zone: Soil Behaviour Type:

1. Sensitive fine grained
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 silty sand
9. Sand
10. Gravelly sand to sand
11. Very stiff fine grained*
12. Sand to clayey sand*

* *Overconsolidated or cemented.*

Figure 5.7 Proposed soil behaviour type classification system from CPTU data (after Robertson *et al.*, 1986).

Roberston's 1990 chart



$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$ $B_q = \frac{u_2 - u_0}{q_t - \sigma_{vo}}$ $F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\%$

Zone Soil behaviour type

1. Sensitive, fine grained;
2. Organic soils-peats;
3. Clays-clay to silty clay;
4. Silt mixtures clayey silt to silty clay
5. Sand mixtures; silty sand to sand silty
6. Sands: clean sands to silty sands
7. Gravelly sand to sand;
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Pore pressure measurements in CPTU

Pore pressure measurements are transient values generated by the undrained shear as the cone is inserted. In coarse grained soils they may be similar to the static pressures but in fine grained soils there may be large excess pore pressures.

Dissipation tests may be carried out if the CPTU insertion is halted.

In a dissipation test the porewater pressure change is obtained by recording the values of the pressure against time during a pause in pushing and while the cone penetrometer is held stationary. It is practical to use either a logarithmic or square root scale for the time factor.

Rate of consolidation parameters may be assessed from the piezocone test using the value t_{50}. In this case, t_{50} is the time for 50% dissipation of excess porewater pressure. The figure (Figure 12) shows one of many charts available for determining the consolidation factor, c_h.

Example of piezocone profile

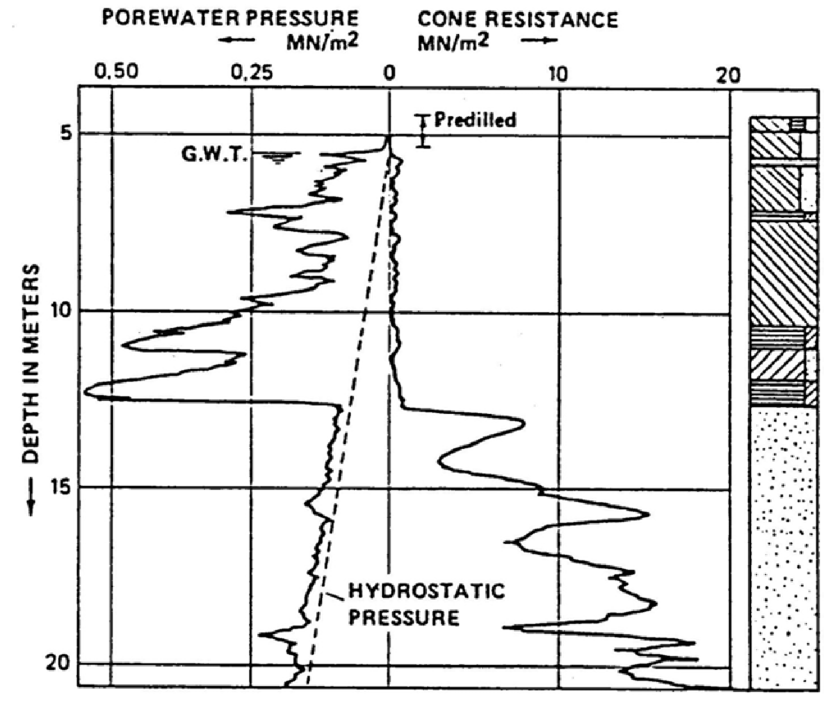


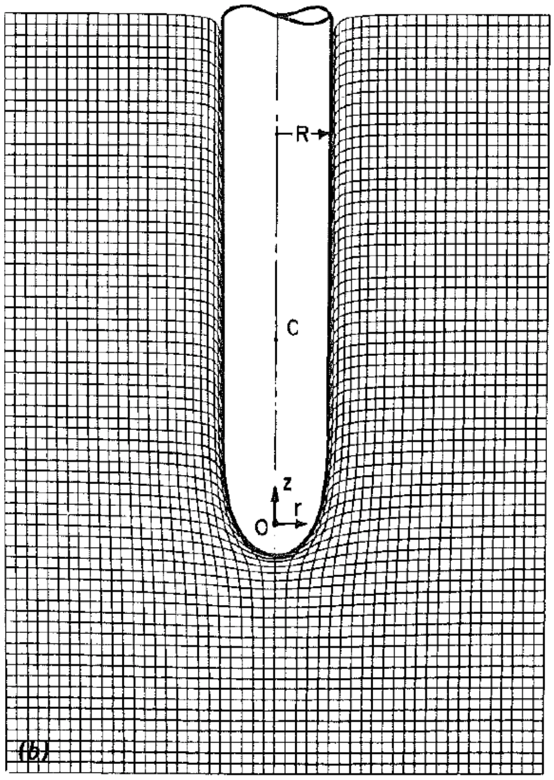
Fig. 9 Typical Piezone Test Result

Penetration of piezocone into clay

When a piezocone is driven into the ground, pore water pressures are set up as a result of the stress changes. The dissipation of these pore pressures occurs in the subsequent consolidation process in a manner dependent on the initial stress distribution as well as on the coefficient of consolidation. The process of analysis therefore has two distinct components. The first is to identify the appropriate total stress distribution caused by driving the piezocone, which acts as the initial condition for the consolidation. The second is to solve the consolidation problem itself.

(Danziger et al. Geotechnique 1997)

A strain path method *(Baligh 1986)* is used to identify the initial stress conditions.



Baligh 1986

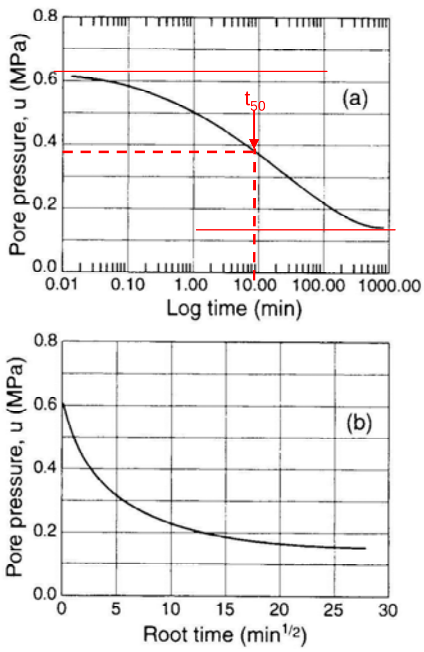


Figure 3.16 Example of pore water pressure decay plotted on (a) log and (b) square root time scales.

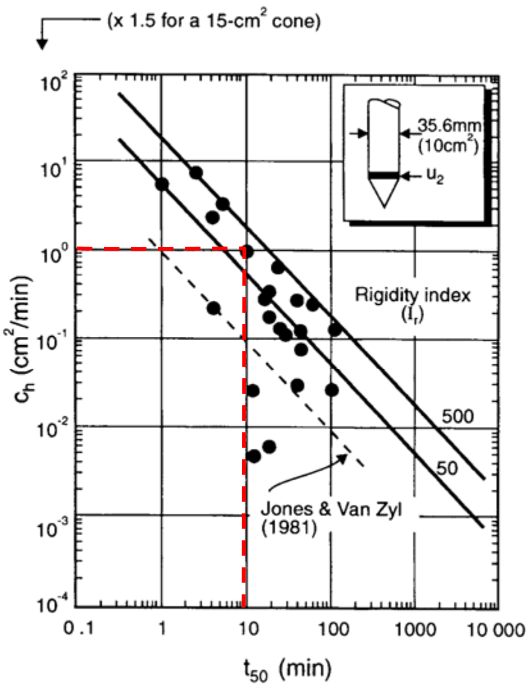


Figure 5.40 Average laboratory c_h values and CPTU results (after Robertson *et al.*, 1992b).

Estimation of the Undrained Shear Strength of Cohesive Soil

The undrained shear strength of cohesive soils can be estimated from the CPT and CPTU with reasonable accuracy. Several authors have presented data in the form of

$c_u = (q_c - \sigma_{vo})/N_k$ [Note that sometimes $c_u = q_n / N_{kT}$ is used]

where σ_{vo} = total overburden stress

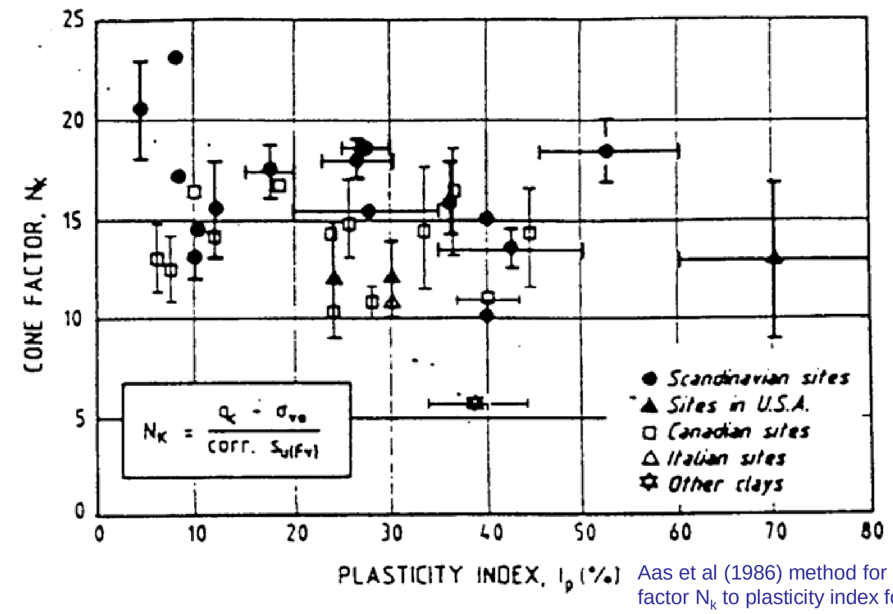
N_k = cone factor

The N_k factor is like a bearing capacity factor, which has been determined theoretically but in practice is determined empirically by correlation of cone resistance to undrained strength measured by vane shear and laboratory tests. Since the undrained shear strength is dependent on the test method, it is important to state what strength is being estimated – eg. field vane shear strength, triaxial compression etc.

Aas et al (1986) present data from several sites and relates N_k to plasticity index, where the cone resistance q_c is correlated with the vane shear strength, and where the vane shear strength has been corrected for strain rate and anisotropy.

It can be seen that the cone factor generally lies between 10 and 20, with the value being independent of the plasticity index. Earlier authors had calculated N_k with uncorrected vane shear strengths, indicating a dependence of N_k on plasticity index.

Factor N_k for correlation of q_c with undrained strength (Aas et al 1986)



Aas et al (1986) method for relating cone factor N_k to plasticity index for CPT

$c_u = (q_t - \sigma_{vo})/ N_{kt}$ where: q_t = corrected cone resistance

N_{kt} = 17 to 18 for normally consolidated (NC) clays

or 20 to 30 for over-consolidated (OC) clays, like London Clay.

Example: Soft clay research site at Bothkennar Geotechnical profile

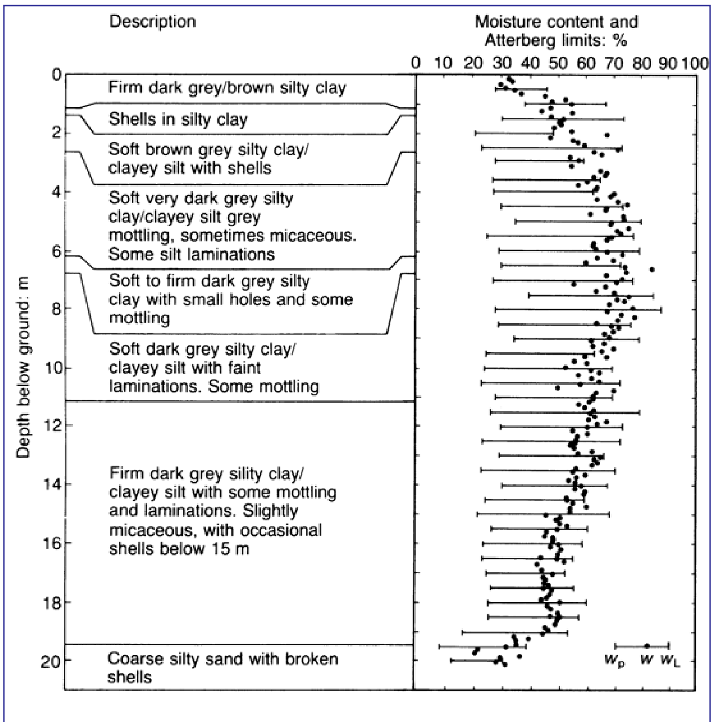
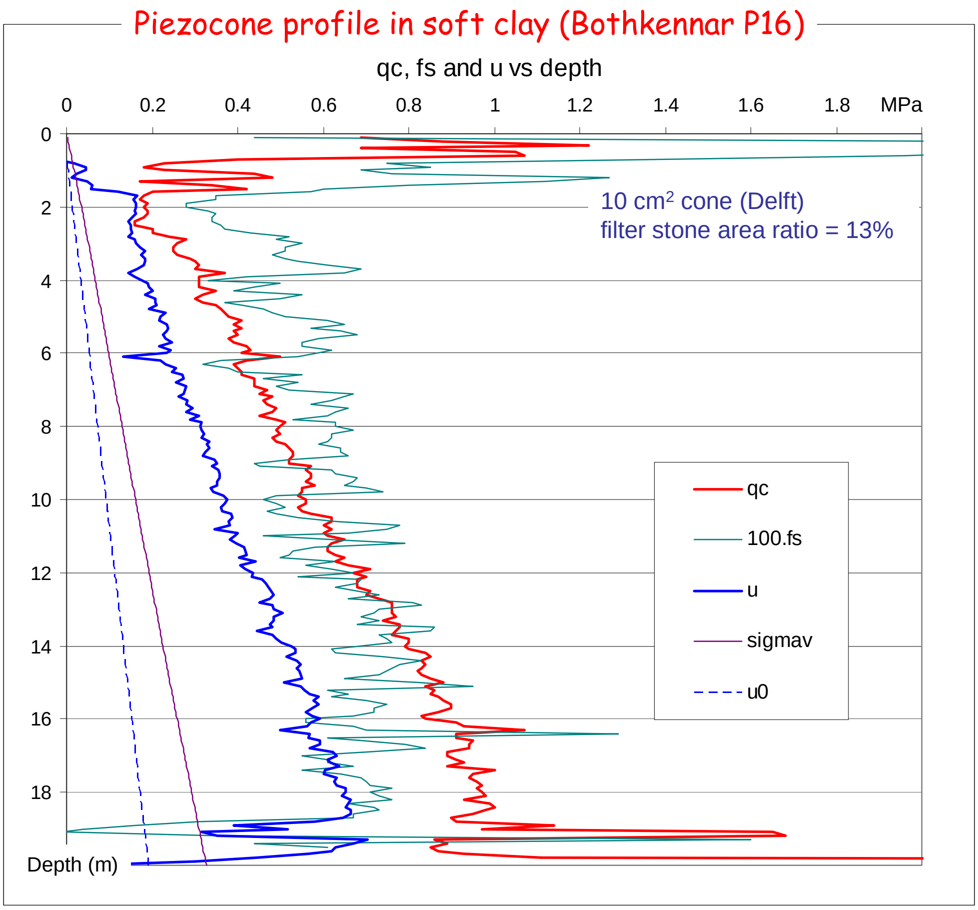
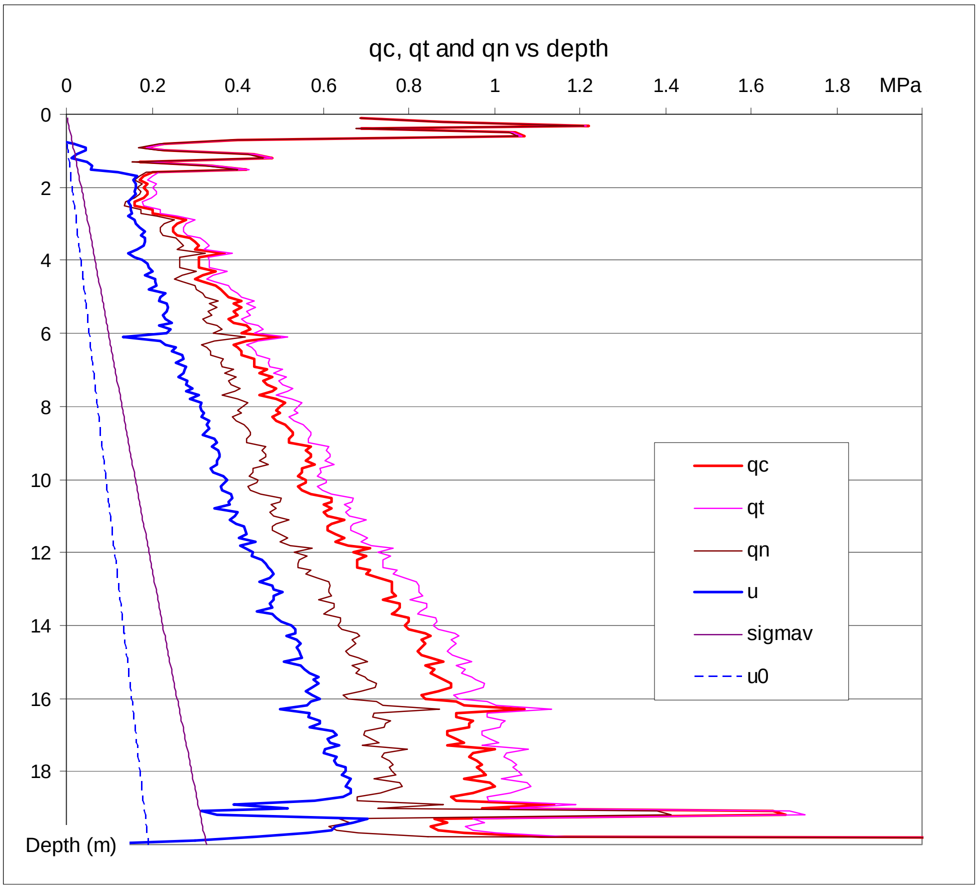


Fig. 4. Geotechnical profile for soft clay in and around borehole D1

- 18 m lightly-overconsolidated soft clay
- c_u = 15 to 60 kPa
- Sensitivity = 5
- OCR = 1.6



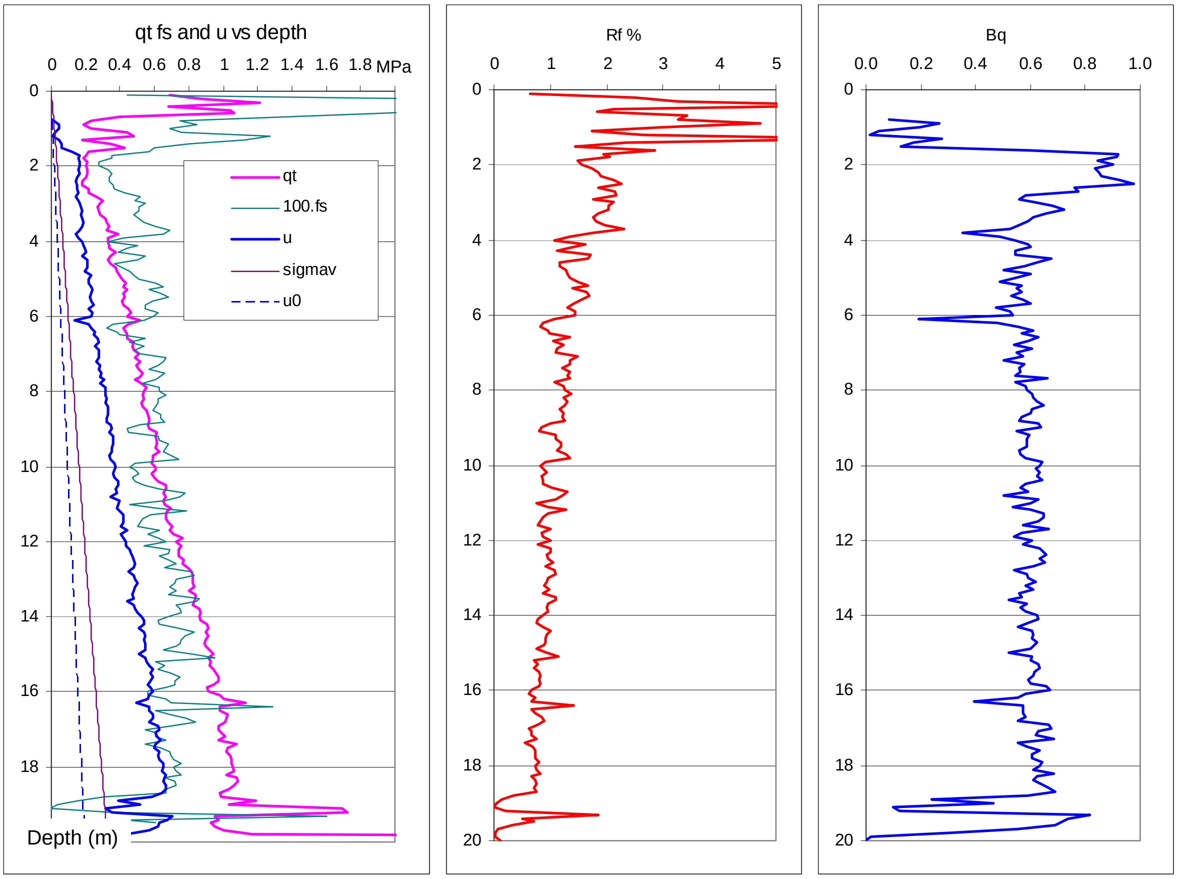


To correct raw cone data use pore pressures u_2 measured behind cone – groove area ratio 13%

at 8m depth:

q_c = MPa, u_2 = MPa so q_t = = MPa

σ_v = MPa so q_n = = MPa



at 8m depth:

f_s = MPa, q_t = MPa so R_f = x100 = %

u = MPa, u_0 = MPa so B_q = =

Example of piezocone profile in soft clay (Bothkennar P16)

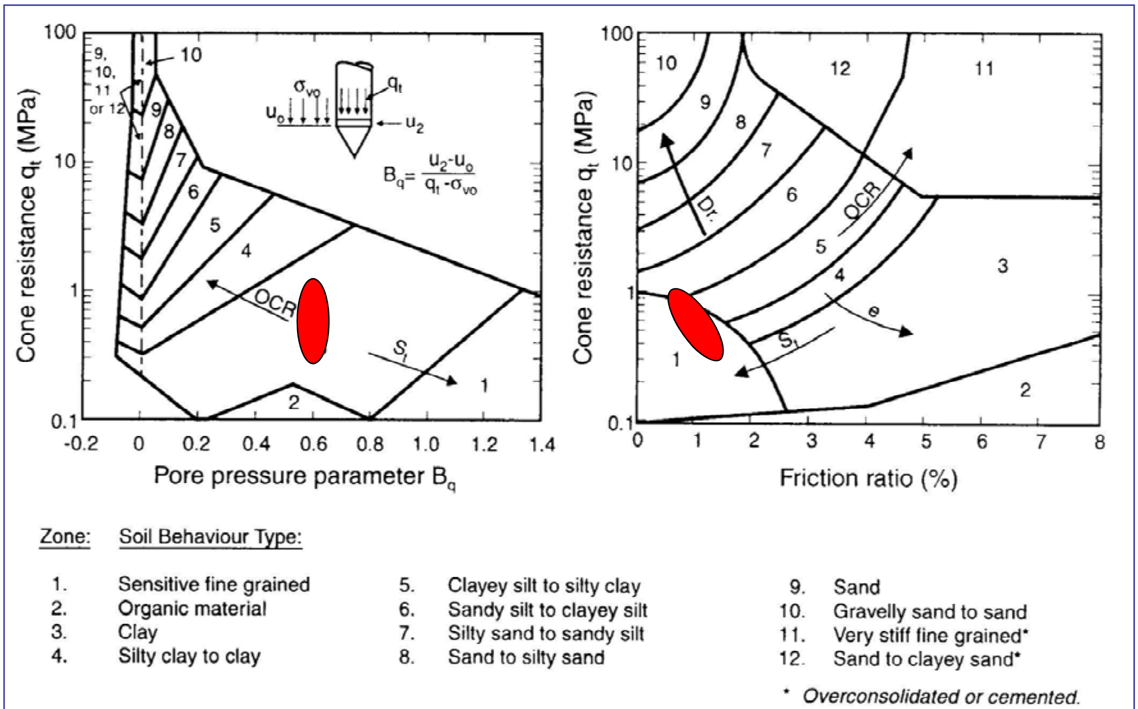
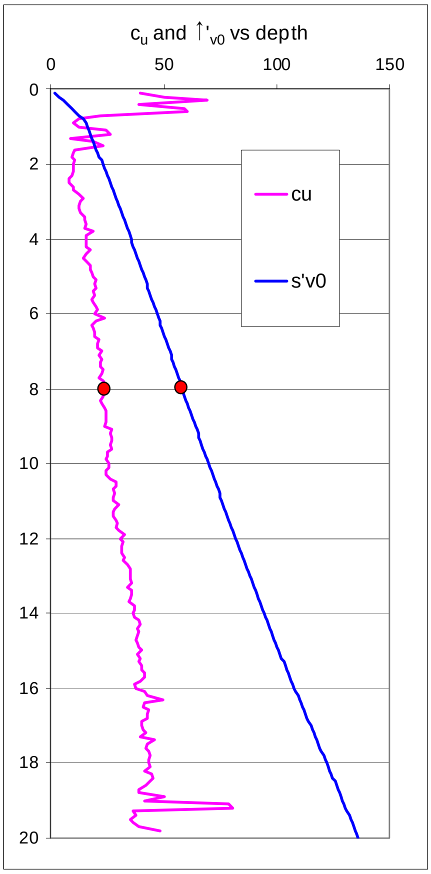


Figure 5.7 Proposed soil behaviour type classification system from CPTU data (after Robertson *et al.*, 1986).

Determination of undrained shear strength from piezocone profile in soft clay (Bothkennar P16)



Undrained strength is obtained from $c_u = (q_t - \sigma_{v0})/N_{kt}$

Using N_{kt} = 17.5 results in the c_u profile opposite

for example at 8m depth q_t = MPa σ_{v0} = MPa

so c_u = = kPa

at that depth σ'_{v0} = kPa

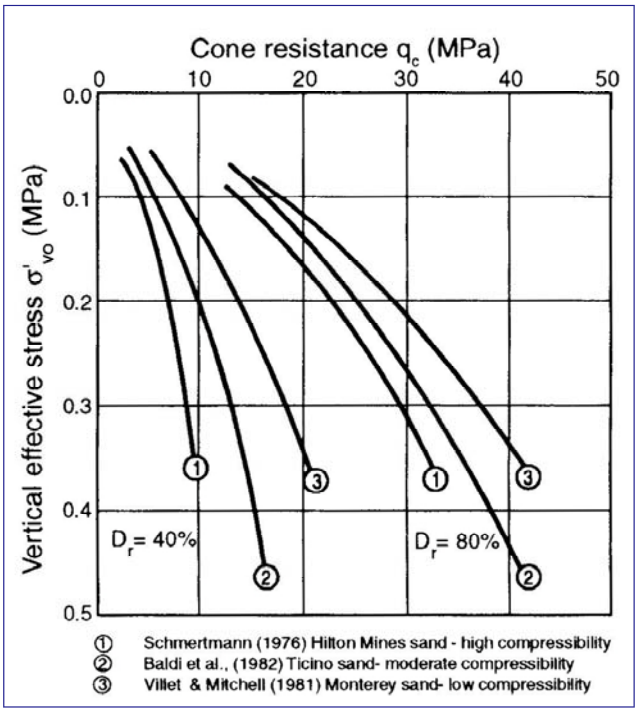
so c_u/σ'_{v0} =

Interpretation of CPT in Cohesionless Soils

The cone resistance depends on –

- relative density;
- current mean stress;
- over-consolidation ratio;
- aging;
- grain-size distribution;
- crushability of soil grains etc.

Various authors have carried out tests in calibration chamber to explore the influence of these as for example shown in the chart opposite.

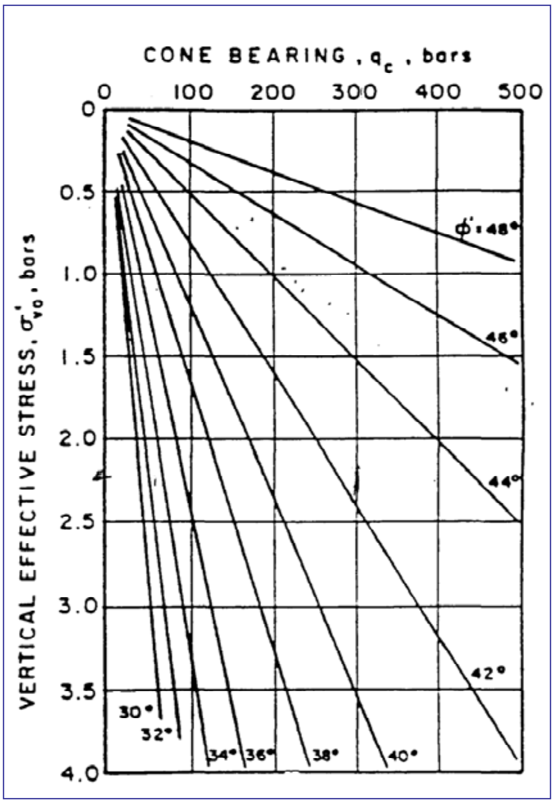


Estimation of the Effective Friction Angle of Cohesionless Soil

The effective friction angle ϕ' of cohesionless soil can be estimated from the CPT or CPTU results using Robertson and Campanella's method (1984).

Campanella and Robertson (1988) indicate that the method

- applies to normally consolidated, uncemented, moderately incompressible, predominantly quartz sands
- for highly overconsolidated sands ϕ' may be up to 2° lower than predicted from the chart
- for highly compressible sand the chart predicts conservatively low friction angles. For the sands included in Robertson and Campanella's analysis the effect could be up to 3°.



Estimation of the Stiffness of Cohesionless Soil

Young's modulus depends mainly on relative density, over-consolidation ratio and current mean stress

Taking Young's modulus $E' = \alpha.q_c$

Schmertman (1970) suggested $\alpha = 2$

Soil type	q_c/N
Silt, sandy silt	2
Clean fine/med sands	3.5
Coarse sands	5
Sandy gravel, gravel	6

Baldi et al (1989) carried out calibration chamber tests to explore the influence of density, stress history etc on α as shown in the chart opposite

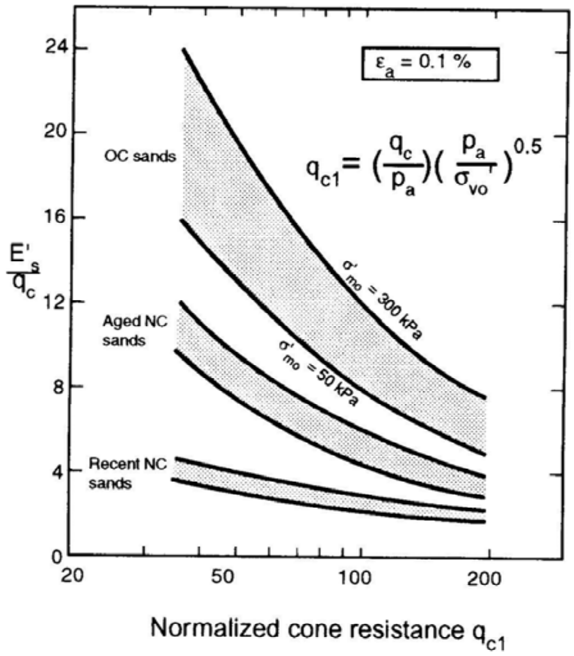


Figure 5.59 Evaluation of drained Young's modulus from CPT for silica sands (from Baldi *et al.*, 1989).

Estimation of settlement of shallow foundations on sand by Schmertmann's (2B - 0.6) strain method (1970, 1978) - 1

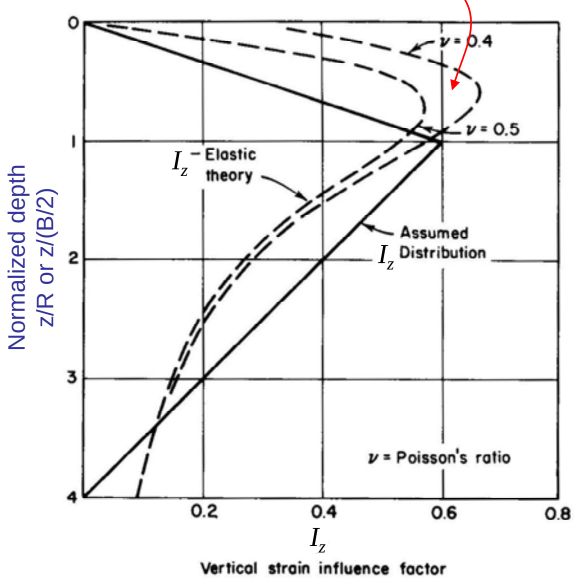
$$\rho = \frac{\Delta p.B}{E} I_\rho$$

Distribution of strain beneath circular or square foundations :

At any level: $\varepsilon_z = \frac{\Delta p}{E} I_z$

So settlement is: $\rho = \sum \varepsilon_z.\Delta z$

peak value $I_z \approx 0.5 - 0.8$



Estimation of settlement of shallow foundations on sand by Schmertmann's (2B - 0.6) strain method (1978) - 2

Meyerhof (1974) $\rho = \dfrac{\Delta p.B}{2q_c}$ roughly equivalent to elastic method taking $E = 2q_c$

Schmertmann's (1970,1978) strain method

settlement $\rho = \sum \varepsilon_z.\Delta z = C_1.C_2.\Delta p.\sum \dfrac{I_z}{C_3 E}.\Delta z$

where $C_1 = 1 - \dfrac{\sigma_1'}{2\Delta p}$ allowance for depth of base

$C_2 = 1 + 0.2\log\dfrac{t}{0.1}$ allowance for creep

$C_3 = 1.25$ for square 1.75 for strip

allowance for axi-sym vs plane strain

ρ = settlement

Δp = net applied stress

σ'_1 = effective stress at foundation level

I_z = strain influence factor

E = soil modulus taken as = $2q_c$ *but see next graph*

t = time in years after loading

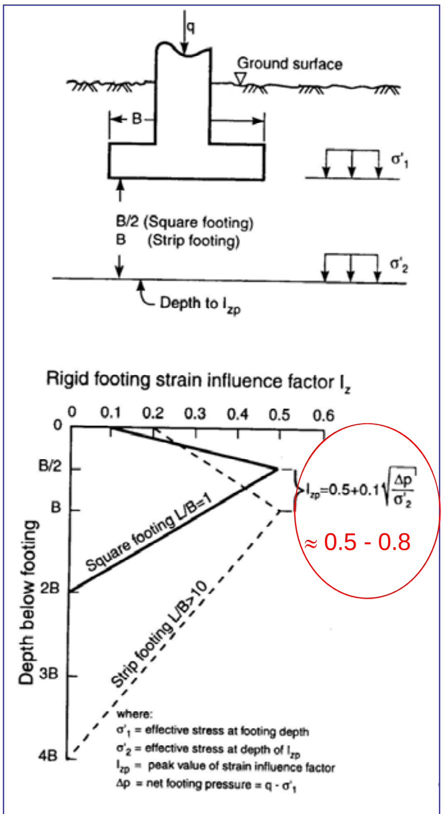


Figure 6.11 Strain Influence method for footings on sand (Schmertmann, 1970).

Example calculation using Schmertmann's method

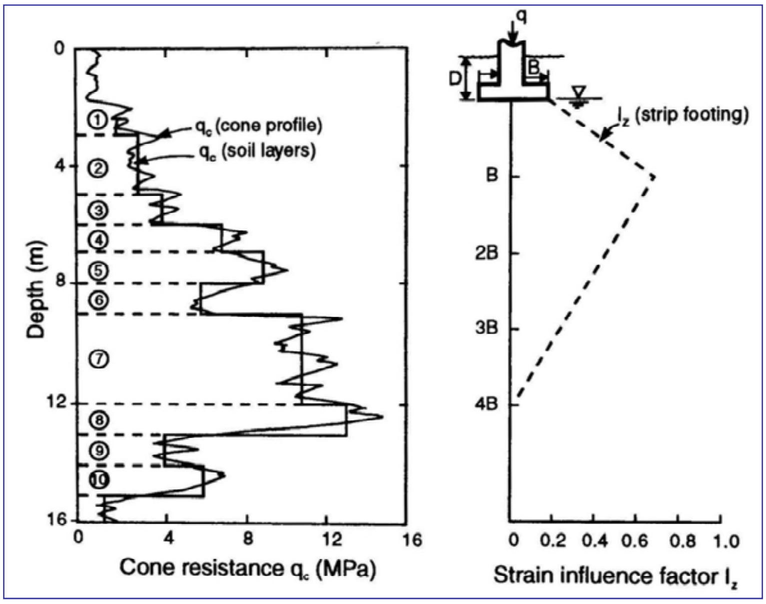
For a strip footing width 3m at 1.8m depth

water at 1.8m depth; soil density 18 kN/m^3

applied stress = 250 kPa

What will be the settlement after 50 years?

- establish depth of influence (2B or 4B) : 12m
- plot I_z vs depth
- divide into layers
- assign average q_c to each layer
- determine E for each layer
- determine I_z at mid point of each layer
- calculate $\Delta p.(I_z/E).\Delta z$ for each layer
- sum
- evaluate C_1, C_2 and C_3
- correct sum to get settlement ρ



layer	top	av depth	z	z/B	Δz	qc	E	Iz	Δp.Iz/E.Δz
1	1.8	2.4	0.6	0.20	1.2	2.0	4.0	0.30	22.6
2	3	3.9	2.1	0.70	1.8	3.0	6.0	0.56	41.7
3	4.8	5.4							
4	6	6.4							
5	6.8	7.4							
6	8	8.4							
7	8.8	10.4							

sum = 119 mm

here I_z max = 0.71

$C_1 = 0.94$, $C_2 = 1.54$, $C_3 = 1.75$

so settlement = 0.94*1.54/1.75*119 = 98mm

Correlation with SPT N-values

A number of studies have been presented over the years to relate the SPT *N*-value to CPT. The figure shows a CPT–SPT correlation with grain size where: q_c = cone resistance (kPa)

p_a = atmospheric pressure (100 kPa)

N_{60} = SPT N-value (energy ratio of about 60%)

D_{50} = mean particle size (mm).

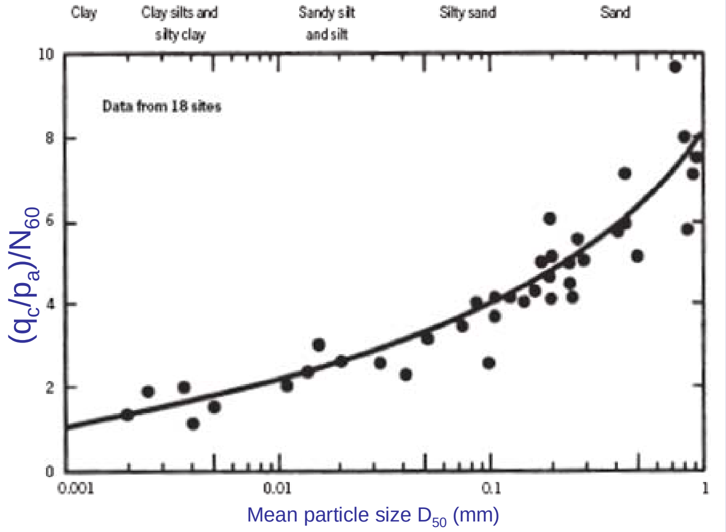


Figure 7 CPT–SPT correlation with grain size (Robertson *et al*, 1983[8])

Sources of Error

There are several sources of potential error in the CPT and CPTU test. These include:

- equipment wear - the cone and friction sleeve may wear and become smaller than standard, and/or smoother or rougher than standard affecting the measured q_c and f_s
- temperature compensation. Electric CPT and CPTU must be temperature compensated and checks made against drift
- improper calibration. Electric CPT and CPTU must be calibrated regularly. The sleeve friction is particularly susceptible to significant error as discussed by Kulhawy and Mayne (1988) and Lunne et al (1986)
- the presence of gravel can affect readings or even halt penetration, and force the cone off vertical which in turn affects q_c and f_s
- use of the wrong capacity cone. In soft loose soils a high load capacity cone may be insufficiently sensitive to measure accurately
- inadequate saturation of the piezometer in the CPTU gives incorrect pore pressures
- failure to note scale change in results. Many instruments change the scale of plot of q_c and sometimes depth when values are large/small. While this is noted on the printout it is sometimes overlooked.

Conclusions

The CPT and CPTU test are very useful versatile both onshore and offshore.

They are used to identify the likely stratigraphy from measured profiles of q_c f_s and u using derived parameters and classification charts.

For cohesive soils –

use correlations to estimate the undrained shear strength c_u and OCR;

determine coefficient of consolidation c_h from dissipation tests.

For cohesionless soils –

use correlations to estimate the relative density, friction angle and stiffness;

assessment of liquefaction potential;

use results directly in pile design and settlement analysis.

With additional sensors the CPT can be used to measure seismic velocities, resistivity, and many other parameters.

Like any in-situ test it is important to correlate data with boreholes and lab testing.