GEOTILL Inc.

Geotechnical Engineering • Subsurface Exploration • Environmental Services • Construction Testing and Material Engineering

GEOTECHNICAL ENGINEERING LIBRARY

GEOTILL

USA

GEOTILL

ENGINEERING, INC.

Phone 317-449-0033 Fax 317- 285-0609

info@geotill.com

Toll Free: 844-GEOTILL

Geotechnical, Environmental and Construction Materials Testing Professionals

www.geotill.com

Offices Covering all USA

CONE PENETRATION TESTING (CPT)

"Simplified Description of the Use and Design Methods for CPTs in Ground Engineering"

The attached notes are a preliminary, simplified description of the interpretation, use and design methods for Cone Penetration Testing (CPTs) in Ground Engineering.

Detailed analysis of CPTs can be a complex subject and a number of papers have been written on this subject. A number of these papers have been summarised to some extent in A.C. Meigh's book "Cone Penetration Testing - Methods and Interpretation" (Ref. 1) and reference should be made to this for a more detailed study of the subject.

It is hoped that the following notes give a simplified introduction to CPTs and takes away some of the myth of the "Black Art"; thereby allowing the average design engineer to appreciate the benefits of CPTs and their use in everyday Ground Engineering working situations.

For further information contact Paul Jacobs
Fugro Limited
18 Frogmore Road
Hemel Hempstead
Herts HP3 9RT
Tel. +44 (0)1442 240781 **www.fugro.co.uk**

TABLE OF CONTENTS

1. INTRODUCTION

1.1. GENERAL

During a CPT, an electrical cone on the end of a series of rods is pushed into the ground at a constant rate of 2cm/s. Continuous measurements are made of resistance to penetration of the cone tip(q_c) and the frictional resistance(f_s), or adhesion, on a surface sleeve set immediately behind the cone end assembly. Measurements can also be made of other soil parameters using more specialised cones such as pore water pressure (piezocone), electrical conductivity, shear wave velocity (seismic cone), pressuremeter cone, etc.

The CPT has three main applications:

1. to determine the soil profile and identify the soils present.

2. to interpolate ground conditions between control boreholes.

3. to evaluate the engineering parameters of the soils and to assess the bearing capacity and settlement of foundations.

In this third role, in relation to certain problems, the evaluation is essentially preliminary in nature, preferably supplemented by borings and by other tests, either *in situ* or in the laboratory. In this respect, the CPT provides guidance on the nature of such additional testing, and helps to determine the positions and levels at which *in situ* tests or sampling should be undertaken. Where the geology is fairly uniform and predictions based on CPT results have been extensively correlated with building performance, the CPT can be used alone in investigation for building foundations.

Even in these circumstances it is preferable that CPTs be accompanied by, or followed by, borings for one or more of the following reasons:

1. to assist where there is difficulty in interpretation of the CPT results.

2. to further investigate layers with relatively low cone resistance.

3. to explore below the maximum depth attainable by CPT.

4. if the project involves excavation, where samples may be required for laboratory testing and knowledge of ground water levels and permeability is needed.

The CPT has four main advantages over the usual combination of boring, sampling and standard penetration testing:

1. It provides a continuous, or virtually continuous record of ground conditions.

2. It avoids the disturbance of the ground associated with boring and sampling, **particularly** that which occurs with the Standard Penetration Test (SPT).

3. It is significantly cheaper.

4. It is faster by a factor of about 10.

Furthermore, the disturbance resulting from the advancement of the cone is consistent between one test and another.

The following sections describe some of the characteristics of CPTs and methods of interpretation of soil parameters, namely:

- soil stratification and estimation of soil type
- soil strength characterisation
- soil deformability characterisation

with associated examples of interpretation where appropriate.

Data from the standard Fugro soil description brochure has been augmented to facilitate interpretation of the differing soil types, and this should be referenced accordingly.

2. SOIL IDENTIFICATION

2.1. GENERAL

When a cone is pushed into the ground the pressure exerted on the end of the cone (cone end resistance 'q_c') is a direct indication of the strength and stiffness of the soil, i.e. it is more difficult to push a cone into a dense sand than, say, a soft clay. This fact is best understood by the analogy of say, pushing a finger, or a wooden stake, into two buckets, one containing sand and the other containing soft clay; this analogy being similar to driving piles into these soils.

This is similar to the Standard Penetration Test (SPT) but in the <u>CPT</u> a <u>continuous</u> resistance profile is available, rather than say tests at 1½-2m depth intervals; and no detrimental ground water effects occur during a <u>CPT</u> compared to an 'SPT'.

As well as measuring the pressure (q_c) on the end of the cone during a test, other measurements can be made which help to identify and classify the soils, two of the most common measurements being:

i) the friction on a cylindrical sleeve (f_s) set immediately behind the cone end; for better classification this friction is related to the cone resistance as a ratio of friction/cone resistance (f_s/q_c), known as the Friction Ratio (R_f).

ii) the pore pressures which are created or induced during insertion of the cone into a cohesive material (pore water pressure "u"). When a granular or more permeable soil layer is penetrated the pore pressure drops as a result of quick drainage; sometimes to as low as the ambient hydrostatic ground water pressure.

Therefore to identify various soil layers these three criteria can be related as follows:

2.2. SOIL CLASSIFICATION

A. **<u>Sand:</u>**

i) insertion of the cone into sand will give a <u>high</u> end resistance

ii) <u>low</u> friction ratio

iii) <u>low</u> pore pressure - quick dissipation of water (high permeability)

B. **<u>Clay:</u>**

i) insertion of the cone into clay will give a <u>low</u> end resistance

ii) <u>high</u> friction ratio

iii) <u>high</u> pore pressure - slow dissipation of water (low permeability)

2.3. EXAMPLES

Using Plate 1 which is a marked up extract of the Fugro Standard Data Sheet:

For Soils "A" and "B" of the **Typical Cone Graph - Figure 1 (Plate 1)**

Compare:

Cone end resistance	(q_c)
Friction ratio	(R_f)
Excess Pore Pressure ratio	(bq)

Then estimating the soil type using the **Guide Chart - Figure 2 (Plate 1)**

Read off the 'q_c' value for the soil strata and the corresponding 'R_f' value and plot these on Fig. 2.

Example

Soil "A"	'q_c' = 8 - 12 MPa	'R_f' = 1%	⇒ **SAND**
Soil "B"	'q_c' = 0.7 MPa	'R_f' = 3-3½ %	⇒ **CLAY**

3. ENGINEERING PARAMETERS

3.1. COHESIVE SOILS (CLAY)

3.1.1. Undrained Shear Strength

a) The preliminary undrained shear strength (Su or Cu) of a clay can be estimated from:

$$Cu = \frac{q_c}{N_{k'}} \qquad (1)$$

where:

q_c = minimum cone end resistance profile values

N_k' = 17-18 for weak normally consolidated (n.c.) clays, e.g.Carse Clays (Grangemouth)

= 20 for overconsolidated (o.c.) clays, e.g. Glacial Tills (Glasgow Boulder Clay).

b) A more detailed undrained shear strength profile can be obtained from:

$$Cu = \frac{q_c - p_o}{N_k} \qquad (2)$$

where:

p_o = overburden pressure
N_k = 15-16 for n.c. clays
= 18-19 for o.c. clays

Example for Soil "B" from Figure 1 - Plate 1 (Fugro Standard Sheet)

q_c at 8m ~ 0.7 MPa (700kPa or 700kN/m^2)

therefore

$$\text{Cu} = \frac{0.7\text{MPa}}{18} = \text{39 kPa, say 40 kPa (40kN/m}^2\text{ - between Soft and Firm constituency).}$$

Note: the shear strength derived is an undrained 'CPT' shear strength and as such should not be considered directly equal to Vane, Triaxial Compression, Pressuremeter, Plane or Simple undrained shear strengths etc., i.e. the appropriate shear strength should be used for the Geotechnical problem being considered.

3.1.2. Deformability/Stiffness

For normally and lightly over consolidated clays (q_c < 1.2 MPa) an "equivalent" coefficient of volume change, m_v, can be derived from the relationship:

$$m_v = \frac{1}{a q_c} \qquad (3)$$

where

a can be derived from Table 3 of Page 48 of Meigh's book dependent on the plasticity, silt and organic content of the soil.

For Grangemouth 'Carse' clays and Glasgow 'Clyde Alluvium' values of ***a = 5 to 7.5*** have been found to be appropriate.

Note: assuming $\alpha = 5$ gives a relatively "conservative" assessment, whereas $a = 7.5$ correlated well in comparative studies for Clyde Alluvium, but could be unconservative in some instances.

It is considered prudent to undertake a sensitivity study of potential settlements, assuming slightly different "α" values to assess the significance of the value adopted.

Example for Soil "B" from Figure 1 - Plate 1 (Fugro Standard Sheet)

with $q_c = 0.7$ MPa

for $a = 5 \quad m_v \cong \frac{1}{5x0.7} = 0.28$ m²/MN

$a = 7.5 \quad m_v \cong \frac{1}{7.5x0.7} = 0.19$ m²/MN

Therefore settlement calculations should be performed using both values of m_v and a sensitivity assessment carried out.

3.2. COHESIONLESS SOILS (SAND/GRANULAR)

3.2.1. <u>Relative Density/Friction Angle</u>

Table 1 of the Fugro standard data sheet (Plate 2) classifies the density of sand related to CPT 'q_c' measurements; and compares these with "SPT - N' value" equivalents.

The relative density *(Dr)* and angle of internal friction *(F')* can also be obtained by direct relationship with this 'q_c' value.

Example for Soil "A" from Figure 1 - Plate 1 (Fugro Standard Sheet)

'$q_{c'}$ at 5 to 7m depth ~ 8-12 MPa

i) Classification is at the higher end of the *MEDIUM DENSE* range **-Table 1 (Plate 2)**

ii) SPT equivalent = $\frac{8}{0.4}$ to $\frac{12}{0.4}$ $\Rightarrow N' = 20 - 30$ - **Table 1 (Plate 2)**

iii) Relative Density *(Dr)* at 6m depth

$s_v' = 6 \times 9 = 54$ kPa (assuming water table @ approx. ground level)

From **Figure 3 - Plate 2 (Fugro Standard Sheet)**

$Dr = 75 - 85\%$

iv) Angle of Internal Friction

From **Table 1 - Plate 2**

$F'_p = 37 - 40°$

Note: The values given are peak values for clean sand. Consideration should be given to a reduction in the F' value used, if F_{cv} is to be considered or if there is a "fines" content, i.e. silt/clay, in the material.

3.2.2. **Deformability/Stiffness**

From correlation studies the following deformation moduli can be derived:

a) Constrained Modulus '*M*'(*or 'D'*) (where '*M*' = $1/m_v$)

b) Elastic Modulus '*E*' (*Young's Modulus*)

c) Shear Modulus '*G*'

This is a relatively complex subject and is dependent on the stress range considered; however, for initial estimates:

a)	$M = 3\,q_c$	***(i.e. m_v equiv. = $1/3\,q_c$)***	***(4)***
b)	$E = 2.5\,q_c$	***(square pad footings - axisymetric)***	***(5a)***
and	$E = 3.5\,q_c$	***(strip footing - plane strain)***	***(5b)***
c)	$G_{ls} = E/2.5$	***(large strains)***	***(6)***

For small strain dynamic studies $G_{ss} \approx 5 \times G_{ls}$ from above (i.e. initial tangent static modulus)

where:

G_{ss} = small strain shear modulus.
G_{ls} = large strain shear modulus.

Example for Soil "A" from Figure 1 - Plate 1 (Fugro Standard Sheet)

where q_c *average* ~ 10 MPa

a) M = 3 x 10 MPa = 30 MPa
$\Rightarrow m_v$ = 0.033 m²/MN

b) E = 2.5 x 10 = 25 MPa for square pad analysis
= 3.5 x 10 = 35 MPa for strip footing analysis

c) G_{ls} = 3 x 10/2.5 = 12 MPa for static analysis
G_{ss} = 5 x 12 = 60 MPa for small strain dynamic analysis.

3.3. ADVANCEMENTS IN CPT'S

Advanced methods of cone penetration testing allow derivation of a number of other ground engineering parameters, e.g.

Piezocone	provides better identification of laminations in soil provides a better estimate of undrained shear strength allows estimation of the coefficient of compressibility - c_h
Conductivity Cone	measures the ground conductivity/resistivity useful for environmental profiling
Thermal Cone	measures ground temperatures up to approx. 100°C useful for environmental profiling
Seismic Cone	allows estimation of small strain dynamic shear modulus - G_{ss}
Pressuremeter Cone	allows better estimation of the soil parameters clay - G, C_u sand - s_{ho}', F', Dr
Fluorescence Cone	determines the presence and concentration of hydrocarbons in the ground.

4. FOUNDATION DESIGN

4.1. GENERAL

Foundation design can be carried out using conventional formulae and the specific soil parameters derived from CPT's. However, there are certain instances when foundations can be designed using the CPT measurements directly.

Frequently CPT's highlight the variability of the underlying soils; compared to conventional intermittent sampling and testing methods, which tend to give a more "average" impression of the ground characteristics. It is important to assess and characterise (possibly averaging) the ground conditions and adopt the appropriate geotechnical design method, i.e. a 2 storey house foundation on medium dense sand may use a relatively simplistic approach to bearing capacity and settlement calculations, compared to a deep bored pile in interlayered loose sands and soft clays.

The following pages give a simplified introduction to some of the CPT design methods, as well as conventional design methods using derived data.

As in all foundation design, it is necessary to consider both the "safe" bearing capacity and "allowable" bearing capacity related to tolerable settlements.

4.2. SHALLOW FOUNDATIONS - STRIPS/PADS

4.2.1. <u>Cohesive Soils</u>

4.2.1.1 Safe Bearing Capacity

Generally, foundation "safe" bearing capacities are based on conventional methods of assessment using the derived undrained shear strength, *Su*, or more commonly *'Cu'*

e.g. the approximate <u>ultimate</u> bearing capacity of a shallow foundation:

$$\textbf{u.b.c} \quad @ \quad \textbf{5.14 x Cu} \qquad \textbf{(7)}$$

resulting in the approximate <u>safe</u> bearing capacity of the foundation:

$$\textbf{s.b.c.} \quad @ \quad \textbf{2 Cu} \qquad \textbf{(8)}$$

(assuming a factor of safety *FoS ~ 2.5 to 3*).

4.2.1.2 Settlement

In general, settlements are estimated using coefficient of volume change (m_v) values derived from the cone end resistance values using equation(3)i.e.:

$$mv = \frac{1}{aq_c}$$

Care has to be taken in the choice of *"a"* value when deriving m_v values, however, a relatively conservative initial assessment can be obtained assuming an *"a"* value of 5.

Thereafter, settlements are estimated using conventional consolidation theory and linear elastic stress distribution methods i.e.:

$$s = Sm_v\, Dp\, h \qquad (9)$$

where:

s	=	estimated settlement
m_v	=	derived m_v value for layer
Dp	=	average stress value for layer from elastic solutions such as Boussinesq, etc.
h	=	layer thickness

4.2.1.3 **Worked Example**

Assuming a strip foundation for a wall line loading of 40 kN/m, constructed at 0.6m depth on "Carse" clay with an upper desiccated crust as shown on Plate 3.

Assess the safe bearing capacity, suitable width and anticipated settlement of the foundation.

Assuming a 1m wide foundation placed on the desiccated crust.

(Care should be taken to check that any weak layer underlying the desiccated crust is not overstressed.)

The stresses and relevant soil parameters below the foundation are detailed in the calculations given on Plate 3.

(i) Check Bearing Capacity

As stated in 4.2.1.1 a foundation width is acceptable if the stress imposed on a soil layer is less than the safe bearing capacity (2Cu) i.e. equation(8):

$$s_Z < 2Cu$$

For the question in hand it can be seen on Plate 3 that the imposed stress from the foundation is less than twice the shear strength of the relative soil layers therefore the foundation size is adequate. If any soil layers are overstressed then a larger foundation width should be adopted and the imposed stress rechecked.

(ii) Settlement Assessment

As given in 4.2.1.2 the settlement of the foundation can be calculated from equation (9) i.e.:

$$s = Sm_V \, Dp \, h$$

From the worked example calculations associated with Plate 3 the cumulative settlement below the foundation as a result of structural loading is less than 25mm, therefore this should be adequate.

4.2.2. **Cohesionless/Granular Soils**

As most granular soils tend to have some variation in their *'qc'* profile, a certain amount of judgement has to be made with regard to "averaging" their profile over depth ranges having "similar" values.

4.2.2.1 **Bearing Capacity**

i) Conventional

Conventional methods can be used (Terzaghi, Brintch-Hansen etc. (Ref.2)) to determine ultimate bearing capacities using derived ∅' values. Thereafter, an appropriate factor of safety (3.0) can be applied to determine a safe bearing capacity.

ii) CPT Method

A CPT 'simplified' method of calculation can be used for foundation design, where the 'safe' bearing capacity (s.b.c.) of a small foundation can be assessed from the equation:

$$\text{s.b.c} = \frac{q_c}{30 \text{ to } 40} \qquad \textbf{(10)}$$

4.2.2.2 Settlement

i) Conventional

Conventional settlement assessment methods can be adopted using *E*, *G* or equivalent SPT *N'* values derived from the CPT 'q_c' profile. Various references such as "Tomlinson"(Ref. 2) or "Burland and Burbridge"(Ref. 3) contain formulae which use these derived parameters and these can be used to assess settlements in the normal manner.

ii) CPT Method

Using 'q_c' values measured directly during CPTs the settlement of a foundation can be assessed as follows:

a) A quick, relatively conservative estimate of settlement of a footing on sand can be obtained directly from 'q_c' values using the equation:

$$s = \frac{p_n B}{2 q_c} \qquad \textbf{(11)}$$

where:

p_n	=	net applied loading(kPa)
q_c	=	average q_c over a depth (kPa)
		equal to B or 1.5B (m), depending on whether pad or strip foundations are adopted
B	=	foundation width(m)
s	=	settlement (m) Note: Use compatible units

b) A more accurate assessment of settlement can be obtained using **Schmertmann's** modified method (Ref. 4), whereby the sand below the foundation is divided into a number of layers, of thickness *Dz*, to a depth below the base of the footing equal to 2B for a square footing and 4B for a long footing (L ≥ 10B).

Settlements are calculated based on the equation:

$$\mathbf{s} \quad = \quad \boldsymbol{C1C2DpS} \left(\frac{Iz}{xq_c} \right) \boldsymbol{Dz} \qquad \textbf{\textit{(12)}}$$

where:

$C1$ = embedment correction = $1\text{-}0.5 \left(\frac{s'_{vo}}{\Delta_p} \right)$

C_2 = creep correction $= 1 + 0.2 \log_{10} (10t)$
t = time in years from load application

s'_{vo} = effective overburden pressure at foundation level

D_p = net foundation pressure (applied pressure (p_n) minus $s_{v'o}$)

I_z = Strain Influence factor from figure 39 of Meigh's book (Plate 5), where the strain distribution diagram is redrawn to correspond to the peak value of I_z obtained from I_{zp} below

Izp = $0.5 + 0.1 \left(\frac{\Delta_p}{s'_{vp}} \right)$ at B/2 for pads
B for strips

s'_{vp} = effective overburden pressure at depth I_{zp}

x = 2.5 for pads
3.5 for strips

Dz = thickness increment.

4.2.2.3 Worked Example

An example of the design of a foundation placed on SAND can be assessed from the typical CPT given on Plate 4.

i) Bearing Capacity

The safe bearing capacity of a foundation placed at 0.6m depth, where $q_c \cong 6.5\text{MPa}$, can be assessed from equation (8):

$$\textbf{s.b.c.} = \frac{q_c}{30 \text{ to } 40}$$

$$\text{s.b.c.} = \frac{6.5\text{ MPa}}{30 \text{ to } 40}$$

$$= 216 \text{ to } 163 \text{ kPa}$$

$\therefore$ adopt s.b.c. of say 175 kPa, but check potential settlements.

ii) Simple Settlement Assessment

For a strip foundation, 1m wide, with a udl = 175 kPa, as shown on Plate 4

B = 1m
p_n = 175 kPa
q_c = 6.5 MPa (as before)

therefore the settlement can be assessed from equation (11) i.e:

$$\mathbf{s} = \frac{p_n B}{2 q_c}$$

$$s = \frac{175 \times 1}{2 \times 6500} = 0.0135\text{m} = 13.5\text{mm}$$

iii) Example of Schmertmann's more accurate method

For the foundation as detailed on Plate 4 and using Plate 5 (the Fig. 39 extract from Meigh's book)

Assuming:

gwt = 1m b.g.l.
g_b = 19 kN/m³,

settlements after a 20 year period and loading, etc. as before

i) $\boldsymbol{s'_{vo}}$ = **0.6m x $\boldsymbol{g_b}$**
= 0.6 x 19 kN/m³ = 11.4 kN/m²

if foundation has backfill above
D_p also=175kPa

however for example, assuming no filling above
D_p=175 - 11.4=164kPa

$$\Rightarrow \boldsymbol{C1} = \boldsymbol{1 - 0.5} \left(\frac{s'_{vo}}{\Delta_p} \right)$$

= 1 - 0.5 (11.4 / 164) = **0.965**

ii) $\boldsymbol{C_2}$ = ***1 + 0.2log$_{10}$(10t)***
= 1 + 0.2 $\log_{10}$ (10 x 20) = **1.46**

iii) $\boldsymbol{s'_{vp}}$ = ***s'_v at B below foundation***
i.e. 1.6m (0.6m +B) below gnd lvl
≅ 1.0 x 19 + 0.6 x 9 = 24.4 kN/m²

$$\Rightarrow \boldsymbol{Izp} = \boldsymbol{0.5 + 0.1} \left(\frac{\Delta_p}{s'_{vp}} \right)$$

= $0.5 + 0.1\ (164 / 24.4)^{0.5}$ = **0.76**
(at B (1m) below foundation)

iv) ***x*** = **3.5 for strip foundations**

v) ***Dz*** = **0.5m** (equal spacing adopted -different thicknesses can be used for certain problems)

With 'q_c' values taken at the appropriate depth increment from the CPT profile
ref. Plate 4

Layer No.	**q_c (MPa)**	**I_z**	**$\frac{Iz}{3.5\ q_c}$ (m^2/MN)**
1	6.6	0.36	0.0156
2	6.6	0.65	0.0281
3	8.0	0.68	0.0243
4	7.0	0.55	0.0224
5	6.5	0.43	0.0189
6	8.0	0.31	0.0111
7	7.2	0.20	0.008
8	6.0	0.06	0.003
			Σ0.1314

Adopting equation 12

$$s = C1\ C2\ Dp\ S\left(\frac{Iz}{xq_c}\right) Dz$$

s = 0.965 x 1.46 x 164 x 0.1314 x 0.5

= 15.2mm after 20 years

or 10.4 mm immediately (i.e. C_2 = 1.0)

compared to 13.5mm immediate settlement by the approximate method.

4.3. DEEP FOUNDATIONS - PILES

4.3.1. <u>General</u>

As stated previously foundations can be designed using conventional formulae and engineering soil parameters derived from CPTs. This is the method of design generally adopted for piled foundations in cohesive soils as discussed below. However, in certain instances, methods of calculating pile capacities directly from the results of CPTs have been developed, mainly for cohesionless soils; an example of a direct method of calculation is given in the relevant section below.

As for all pile design there are numerous methods of calculation for varying pile types in different ground conditions; far too numerous to mention in a document such as this. Examples of 'general' methods of calculation are given below for guidance and these should give a reasonable estimate of pile capacity in certain ground conditions, however, as for all pile design, these should be confirmed by a series of load tests on site, possibly comprising both static and dynamic methods.

In addition, where relevant the capacity and behaviour of pile groups should be assessed using conventional methods of analysis using soil parameters and properties derived from CPT results, i.e. E, n, Cu, m_v, c_h, etc.

4.3.2. Cohesive Soils

There are methods of calculating the bearing capacity of piles in clay in terms of effective stress parameters, however, it is more common to adopt total stress methods using the undrained shear strength *'Cu'*. At present there are no commonly adopted procedures for determining pile capacities in clay directly from CPT methods and, as such, the general methods used are those in which *'Cu'* is obtained from the CPT results and used in "standard" formulae such as those given by Tomlinson (Ref. 2).

4.3.2.1 Bored Piles

The ultimate bearing capacity *(Qt)* of a pile bored into clay may be expressed as:

$$Qt = aC\bar{u}As + NcCuAb \qquad \textbf{(13)}$$

where:

a	=	Adhesion factor derived from empirical relationships with shear strength (different from α value in equation (3))
$C\bar{u}$	=	mean undrained cohesion over length of shaft considered
As	=	pile shaft area
Nc	=	bearing capacity factor (~ 9.0)
Cu	=	undrained cohesion of pile base
Ab	=	base area

Compared to driven piles a reduced μ value is generally adopted because of:

i) softening of bore walls
ii) seepage of water into the bore
iii) moisture and air absorption if concreting is delayed.

On this basis an μ value of 0.45 is generally adopted for conventional shell and auger, open bore, type construction. However, with the more modern advanced, closed bore, type construction methods, e.g. CFA or Atlas, there is less likelihood of clay softening and/or time delays and, as such, there is a school of thought that an μ value of the order of 0.6 may be more appropriate.

Note: A limiting μCu value of 100kPa is recommended by Skempton for piles in London Clay (Ref. 5), whilst a maximum adhesion value of 70kPa is considered appropriate for Glacial clays with a shear strength in the range of 80 to 200 kPa (Ref 6).

4.3.2.2 Driven Piles

The ultimate bearing capacity *(Qt)* of a pile driven into clay may be expressed as:

$$Qt = \alpha C\bar{u}\,As + NcCuAb \qquad (14)$$

where:

α = Adhesion factor derived from empirical relationship with shear strength - see Plate 6

$C\bar{u}$ = mean undrained cohesion over length of shaft considered

As = pile shaft area

Nc = bearing capacity factor (~ 9.0)

Cu = undrained cohesion of pile base

Ab = base area

Note - Similar to bored piles, a limiting value of αCu of 85kPa is recommended for piles driven into Glacial clays with a shear strength in the range of 80 to 200 kPa (Ref. 6).

4.3.3. **<u>Cohesionless Soils</u>**

The capacity of piles constructed in cohesionless soils can be derived using conventional formulae and soil parameters derived from CPTs. However, in certain instances, mainly driven piles, the capacity can be derived directly from the CPT values, an example of which has been given below.

4.3.3.1 **Bored Piles**

(i) Shaft Friction

The ultimate shaft friction *(Qs)* of a pile in cohesionless soil can be calculated from the equation

$$Qs = K\bar{p}_0' \tan d As \qquad (15)$$

where:

K = an earth pressure coefficient related to the initial soil stress history and the modifying effects of pile construction on the stress fields. *

= 0.7 for normal bored piles

= 0.9 for CFA piles in clean sand

$\bar{p}_o'$ = the effective overburden pressure at the depth considered

δ = effective angle of skin friction between the pile and the soil with δ normally assumed to be = F'

where F' = effective angle of internal friction for the soil.

As = area of pile shaft.

* If pile construction is poor the *K* value may drop to *Ka* soil conditions, i.e. 0.3 to 0.4 but this may be restored to a degree by hydrostatic concrete pressures during placement.

(ii) End Bearing

The ultimate end bearing *(Qb)* of a pile in cohesionless soil can be calculated from the formula

$$Qb = Nqp_o' Ab \qquad (16)$$

where:

Nq = bearing capacity factor (commonly Berezantsev is used as given on Plate 7).

p_o' = effective overburden pressure at base

Ab = area of pile base

* Allowance for the pile toe "depth of embedment" into competent strata should be made. Consideration should also be given to a reduction in F', and thus Nq, if the construction method loosens the soil base, i.e. shell and auger compared to CFA.

4.3.3.2 Driven Piles

(i) Shaft Friction

The ultimate shaft resistance *(Qs)* of a driven pile in cohesionless soil is more difficult to determine and research continues in this field. Some research (Vesic - Ref. 7) indicates that once a pile reaches a certain depth of embedment (15 to 20 diameters) the shaft resistance approaches a constant ultimate value.

Some practitioners adopt the same formula for the shaft resistance of a driven pile as that for a bored pile in cohesionless soil i.e.:

$$Qs = Kp_o' \tan d As \qquad (17)$$

except that they use a higher value of *K* (1.0 to 2.0) due to the densification effect of a pile on the surrounding soils during driving.

However, the following equation has been found to provide a reasonable estimate of ultimate shaft friction for single piles of lengths up to 15m, driven into normally consolidated sand, directly from CPT results.

$$Qs = \frac{q_{CS} As}{200} \qquad (18)$$

where:

q_{cs} = average q_C within the depth of embedment

As = area of embedded pile shaft

The denominator value of 200 has been found to be suitable for precast concrete piles driven into the silty sands of the Clyde Alluvium, whereas other values have been proposed for different conditions, as given in Meigh's book and summarised below:

Pile Type	Soil	Denominator	Author
Precast concrete	Silt	140	Thorburn
Open ended Steel Tube	Sand	300	Te Kamp
Timber	Sand	80	Meigh
Precast concrete	Sand	80	Meigh
Steel, displacement	Sand	80	Meigh
Steel, open tube	Sand	125	Meigh

Note - A limiting value of 120 kPa is recommended for *Qs* in all situations (Ref. 1).

(ii) End Bearing

The ultimate end bearing capacity *(Qb)* of a pile driven at least 8 diameters into a uniform sand deposit is generally equal to the cone resistance, and can be calculated from the formula:

$$Qb = (0.25\, q_{c0} + 0.25 q_{c1} + 0.5\, q_{c2})\, Ab \qquad (19)$$

where:

q_{c0} = average q_c over a distance of 2 pile diameters below the pile base

q_{c1} = minimum q_c over same distance

q_{c2} = average of the minimum q_c over a distance of 8 pile diameters above the pile base, ignoring any value greater than q_{c1}, also ignore any local peak depressions in sand

Ab = area of pile base.

If the pile is driven only 1 or 2 diameters into a fine grained cohesionless soil due to a very dense layer, or enlarged bases, and the q_c reduces within 3.5 pile diameters below the base, then a more appropriate equation for this shallow embedment condition is:

$$Qb = (0.5 q_{cb} + 0.5\, q_{ca})\, Ab \qquad (20)$$

where:

q_{cb} is the average cone resistance over a distance of 3.5 diameters below the base and can be determined from:

$$\mathbf{qcb} = \frac{(q_{c1}, q_{c2} \ldots\ldots q_{cn}) + nq}{2n}$$

where:

q_{c1}, q_{c2}, ----- q_{cn} = cone resistance at regular intervals to a depth of 3.5 diameters and q_{cn} is the lowest resistance within this depth. The number of measurements is *n*.

$\boldsymbol{q_{ca}}$ = average q_c over a distance of 8 pile diameters above the base, neglecting any values greater than q_{cn}.

For intermediate penetrations between 2 and 8 equivalent pile diameters it is reasonable to interpolate linearly between the shallow and deep conditions above.

As for any design when layered soil conditions exist, i.e. sand, silt and clay, special consideration of the various capacities and interaction should be made, with more emphasis put on the interpretation of load test results.

Typical Dutch practice is to limit the value of q_c used (normally to 30MPa) and to limit the ultimate end bearing capacity (*Qb*) to a value not exceeding 15MPa.

The above method of calculation appears complex on first impression, however, in reality it is reasonably easy. The example given later illustrates the methodology of this method of analysis.

4.3.4. <u>Allowable Capacity for Piles</u>

The allowable working load of a pile (Q_{all}) is equal to the sum of the shaft friction and base resistance divided by a suitable factor of safety. In general, a factor of safety of 2.5 is adopted which results in the equation:

$$\textbf{Qall} = \frac{Qs' + Qb}{2.5} \qquad \textbf{(21)}$$

where:

Q_s' is the ultimate skin friction calculated using the <u>average</u> shear strength.

Also Q_{all} should be controlled such that:

$$\textbf{Qall} < \frac{Qs''}{1.5} + \frac{Qb}{3.0} \qquad \textbf{(22)}$$

where:

Q_s'' is the ultimate skin friction calculated using the <u>lowest</u> range of shear strength.

It is reasonable to take a safety factor equal to 1.5 for the skin friction because the skin friction on nominal sized piles is generally obtained at small settlements, i.e. 3 to 8mm, whereas the base resistance requires a greater settlement for full mobilisation i.e. 25 to 50mm, as detailed later.

4.3.5. Settlement

There are no methods of calculating the settlement of a pile, or a pile group, directly from CPT results and 'normal' methods of assessment are generally adopted using parameters derived from CPT results i.e. E, n, Cu, m_v, c_h etc.

Some methods of general and detailed settlement assessment are given below:

Method 1:

In general, for a relatively standard pile design, if a factor of safety of 2 .5 is adopted, pile settlements at working load should be of the order of 1 to 2% of the pile diameter, due to skin friction being fully mobilised at this deflection i.e.

$$s = 1 - 2\%\ d_b \tag{23}$$

where:

d_b	=	diameter of pile base e.g. 5-10 mm for a 450mm ∅ pile.

Method 2:

The end bearing capacity may require movements of the order of 10 to 20% of the pile diameter to be fully mobilised. Therefore if the working load capacity relies on a significant amount of end bearing, pile settlements at working load will generally be proportional to the load mobilised i.e.

$$s = \frac{Q_m}{Q_b}\ (10 to 20\%) d_b \tag{24}$$

where:

Q_m	=	Amount of working load derived from end bearing
Q_b	=	Ultimate end bearing capacity

Method 3:

An approximate estimate of the settlement of a single pile in sand can be obtained from Meyerhof's (Ref. 8) equation:

$$s \approx \frac{d_b}{30F} \tag{25}$$

where:

F	=	factor of safety on ultimate load (>3).

Method 4:

For dense soils and relatively undisturbed pile bases the settlement of a pile can be assessed from the expression:

$$s = \frac{p}{4}\frac{q}{E}d\left(1-v^2\right)f \qquad (26)$$

where:

E	=	soil modulus of "elasticity" (from the CPT 'q_c' value)
q	=	applied base pressure
s	=	settlement
d	=	pile diameter
ν	=	poissons ratio (say 0.3)
f	=	depth factor (0.5 for deep piles).

4.3.6. **Group Analysis**

The behaviour of a pile group can also be assessed taking into account factors such as interaction (loosening/densification), spacing, underlying compressible layer, frictional or end bearing load transfer mechanisms etc.

4.3.7. **Special Conditions**

Some special conditions, peculiar to piling, such as negative skin friction forces etc. may have to be considered in the overall design, however these require specialist geotechnical input and are not addressed in this document; not being directly relevant to CPTs.

4.3.8. **Worked Example**

Assess the capacity of a 250mm square precast concrete pile, driven to around 9m depth in the soil given on Plate 8.

The depth of embedment of the pile toe into the main sand layer
= 9m - 6.4m = 2.6m

$\Rightarrow \frac{2.6}{0.25}$ = 10 pile diameters (10 d)

> 8D $\therefore$ deep embedment design method appropriate

(i) Shaft Resistance - Equation (18)

$$Qs = \frac{q_{cs} As}{200} = \frac{18000x4x0.25x2.6}{200} = 234kN$$

from CPT plot

q_{csmin}	=	14MPa
q_{csmax}	=	20-22MPa
$q_{csaverage}$	=	18MPa = 18000kN

(ii) Base Resistance - Equation (19)

$$\mathbf{Q_b = (0.25q_{c0} + 0.25q_{c1} + 0.5q_{c2})Ab}$$

= (0.25 x 17750 + 0.25 x 17000 + 0.5 x 17000) x 0.25 x 0.25

= **1074kN**

from CPT plot

q_{c0}	=	17.75MPa
q_{c1}	=	17.0MPa
q_{c2}	=	17.0MPa

Total ultimate capacity *Qt* = 234 + 1074 **= 1308kN**

Working capacity Q_{all} = $\frac{1308}{2.5}$ **= 523 kN**

say <u>500kN</u> (50 tonnes)

(iii) Settlement

Method 1

Full skin friction is generally mobilised at pile vertical movements of around 1 to 2% of the pile shaft diameter.

Therefore using equation (23)

and assuming $d_b = \sqrt{(0.25x0.25x\frac{4}{p})} = 0.28m$

$$s = 1 - 2\%\ d_b$$

$$= 3 \text{ to } 6mm$$

Method 2

Full end bearing is mobilised at pile toe movements of 10-20% of the pile diameter, i.e. Qb mobilised at 30 to 60mm for 250mm square pile.

At working load, the end bearing load from capacity calculations above

$$Q_m = 500 - 234kN = 266kN$$

this is proportionally 266/1074 = 25% of the ultimate base capacity

$\therefore$ the movement of the pile head ~ 7 to 15mm at working load
i.e. 25% of 30 to 60mm.

Method (3)

Using equation (25)

$$\mathbf{s} = \frac{d_b}{30F}$$

$$= \frac{250}{30x2.5} = 3.5\text{mm}$$

Method (4)

Using equation (26)

$$s = \frac{p}{4}\frac{q}{E}D(1 - v^2)f$$

$q = \frac{266}{0.25x0.25}$	=	4256kPa
d_b	=	0.28m
f	=	0.5
ν	=	0.3

at 9m depth

q_{c1}	=	17MPa
σ_{v0}	=	9x10 = 90kPa
E_{50}	=	20MPa (from fig.17 of Meigh's book - Plate 9)

$$\mathbf{s} = 0.785\frac{4256}{20E^3} \text{ x } .28\ (1-.3^2).5 = \mathbf{21mm}$$

Special consideration may also have to be given to:

a) reduced capacity if the toe extended further towards the weaker zones below

b) possible negative skin friction of the upper soils if ground levels were increased

c) consolidation of the underlying weak layers due to pile group action i.e. large loaded area

Capacity

From the above it can be seen that a 250mm square precast concrete pile driven to 9m at the site in question would have a working load capacity of the order of 500kN.

Settlement

It can be seen that the settlement at working load is estimated to be between 5mm and 20mm dependent on the method of analysis. It is obvious that the actual settlement of the pile is difficult to determine accurately and is best assessed from maintained load tests in the field; however, it is estimated this will be of the order of 10 to 15mm.

Dynamic load testing of piles can give a reasonable indication of load capacities and anticipated settlements.

5. REFERENCES

1. Meigh, A.C., 1987 "Cone Penetration Testing - Methods and Interpretation", CIRIA, Butterworths.

2. Tomlinson, M.J., "Foundation Design and Construction", Pitman.

3. Burland, J.B. and Burbridge, M.C., "Settlement of Foundations on Sand and Gravel" I.C.E., Glasgow and West of Scotland Association Centenary Lectures, 1984.

4. Schmertmann, J.H., Hartman, J.P., and Brown, P.R., 1978 "Improved Strain Influence Factor Diagrams", Proc. Am.Soc.Civ. Engrs - J. Geotech, Engineering Division August 1978, 104 (GT8), 1131 - 1135.

5. Skempton, A.W., "Cast In-Situ Bored Piles in London Clay" Geotechnique, 9, No. 4, pp 153 - 173 (December 1959).

6. Weltman, A.J. and Healy, P.R. "Piling in 'Boulder Clay' and other Glacial Tills", CIRIA Report PG5, 1978.

7. Vesic, A.S., "Tests on Instrumented Piles, Ogeechee River Site", J. Soil Mech. and Foundation Div., Amer.Soc.Civ.Eng., 96, SM2, (1966).

8. Meyerhof, C.G. (1959)" Compaction of Sands and Bearing Capacity of Piles". Proc.Am. Soc.Civ. Engrs. - J. Soil.Mech.Found.Div., Dec. 1959, 85 (SM6), 1-30.

APPENDIX

Interpretation of Static Cone Penetration Tests

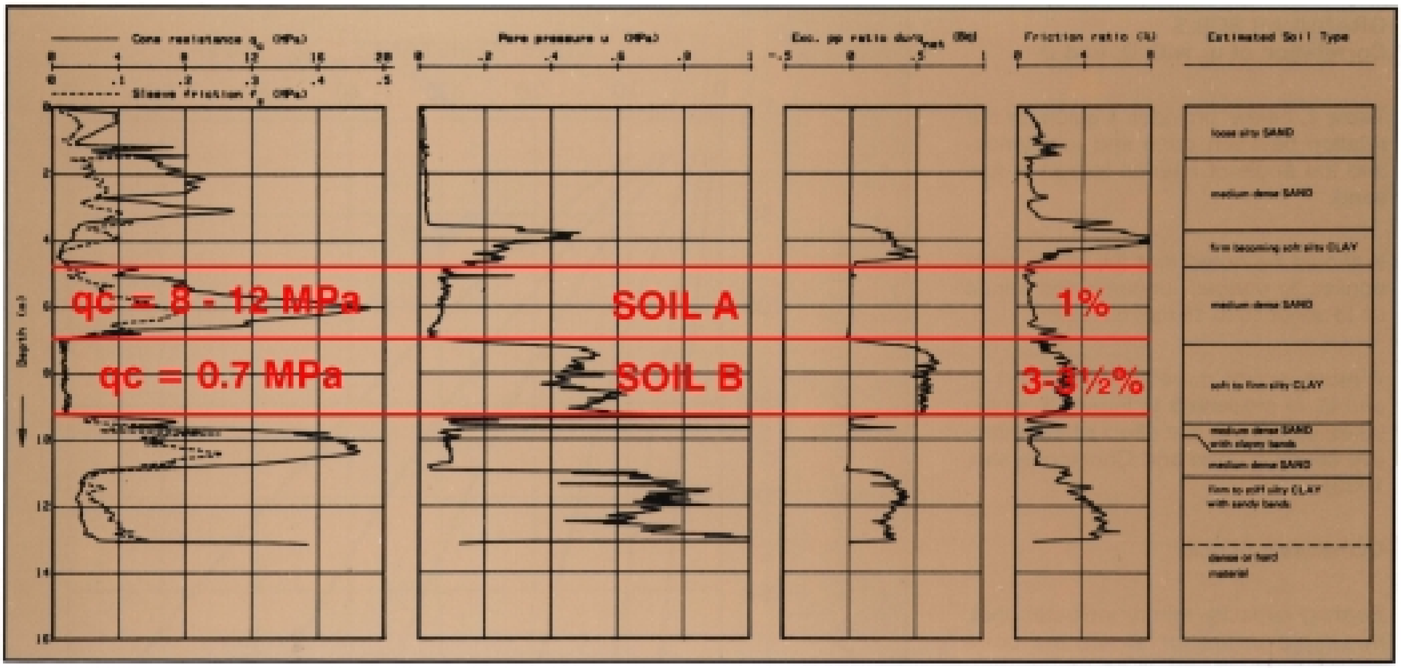


Figure 1 - Typical Cone Test Graph With Estimated Soil Type

INTERPRETATION OF STATIC CONE PENETRATION TESTS BY USE OF THE FRICTION RATIO

Extensive research has indicated that the ratio of local side friction to cone end resistance ("friction ratio") assists in identifying the soil type. The results of various research studies (Meigh 1987) have been produced in graphical form and a modified version for British soils is presented in Figure 2, where the soil type is given as a function of cone end resistance and friction ratio.

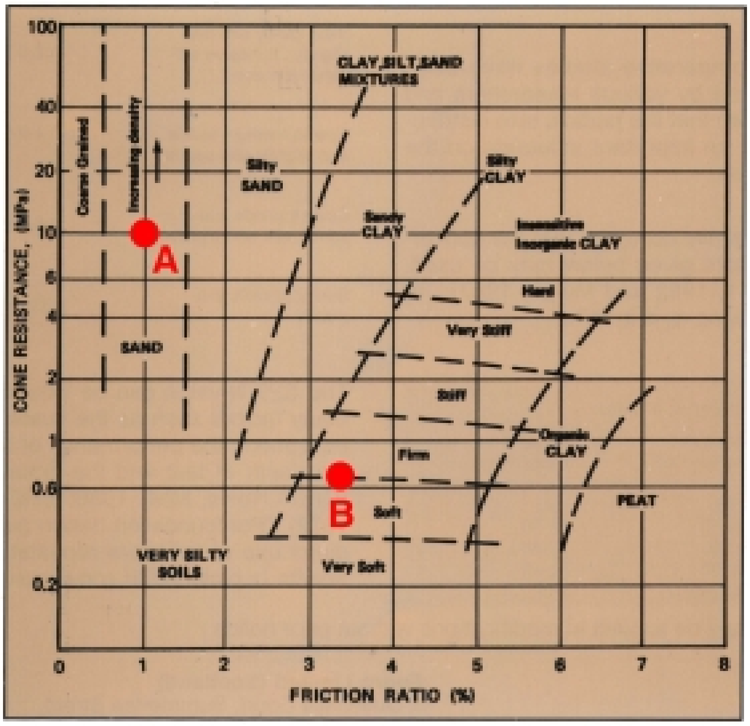


Figure 2 - Guide For Estimating Soil Type

THE USE OF PORE PRESSURE READINGS

The additional measurement of pore pressure with the piezocone assists in identifying the soil types.

Variations in pore pressure reflect changes in stratification that cannot always be determined with q_c or f_s. For instance, in Figure 1, changes in pore pressure response in the clay layers indicate the permeable seams or lenses that can greatly influence the drainage characteristics of the stratum.

Furthermore, the excess pore pressure ratio could give an indication of the stress history of the soil.

GRANULAR SOILS
Correlation of q_c with D_r and Ø

Table 1, below, provides a guide for the relation between cone end resistance and the angle of internal friction in fine sand.

It should be noted that the guide table applies to unaged, uncemented sands up to about 10 to 15 metres depth.

A more recent correlation between q_c and D_r is presented in Figure 3, which takes account of the effect of the effective stress (Lunne and Christophersen, 1983)

COHESIVE SOILS

Bearing capacity theory indicates that, in simple terms for Ø =0, the cone resistance q_c should be related to overburden pressure p_o and undrained shear strength s_u in the following way (Sanglerat et al 1972):

$$q_c = s_u N_k + p_o \qquad (1)$$

where N_k is a bearing capacity factor or "cone factor". However, in some circumstances, Fugro use a modified expression in which the effect of overburden pressure is included in the cone factor.

SOIL B

$$q_c = s_u N'_k \qquad (2)$$

To use either equation, the cone factor must be determined empirically, or be known from correlations based on previous investigations in the same clay. The value of the cone factor depends on the stress-strain properties of the clay and is frequently found to lie in the range 15 to 20, although it should be noted that values outside this range have been observed.

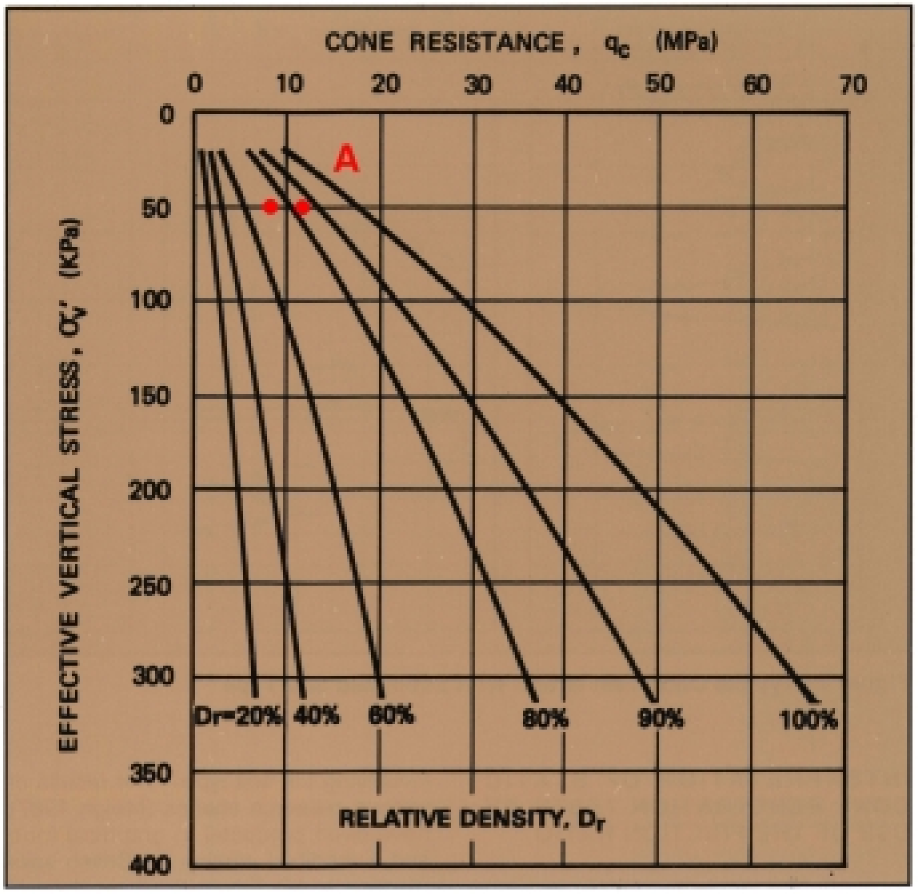


Figure 3 - Suggested Relationship between σ_v', q_c and D_r for unaged uncemented fine to medium quartz sand

CORRELATION OF q_c WITH SPT-'N' VALUE

Many comparative studies have been carried out by various researchers and it is known that the particle size distribution has an important influence on the correlation.

It is generally accepted that the conversion factors given below may be used. (ESOPT1, 1982 and Meigh, 1987). In the following q_c is in MPa

Soil Description	q_c / N
Silts, sandy silts and slightly cohesive silt-sand mixtures	0.2-0.3
Fine to medium sands and slightly silty sands	0.4-0.5
Coarse sands and sands with some gravel	0.6-1.0
Sandy gravels and gravel	1.1-1.8

The SPT N-value can be influenced by many factors such as the quality of the equipment, the performance of the test, the depth of test and the groundwater (CIRIA News, No4, 1986; BRE Report 1979). "For foundation design purposes, direct use of the more repeatable CPT results is preferred to conversion to SPT

SOIL A

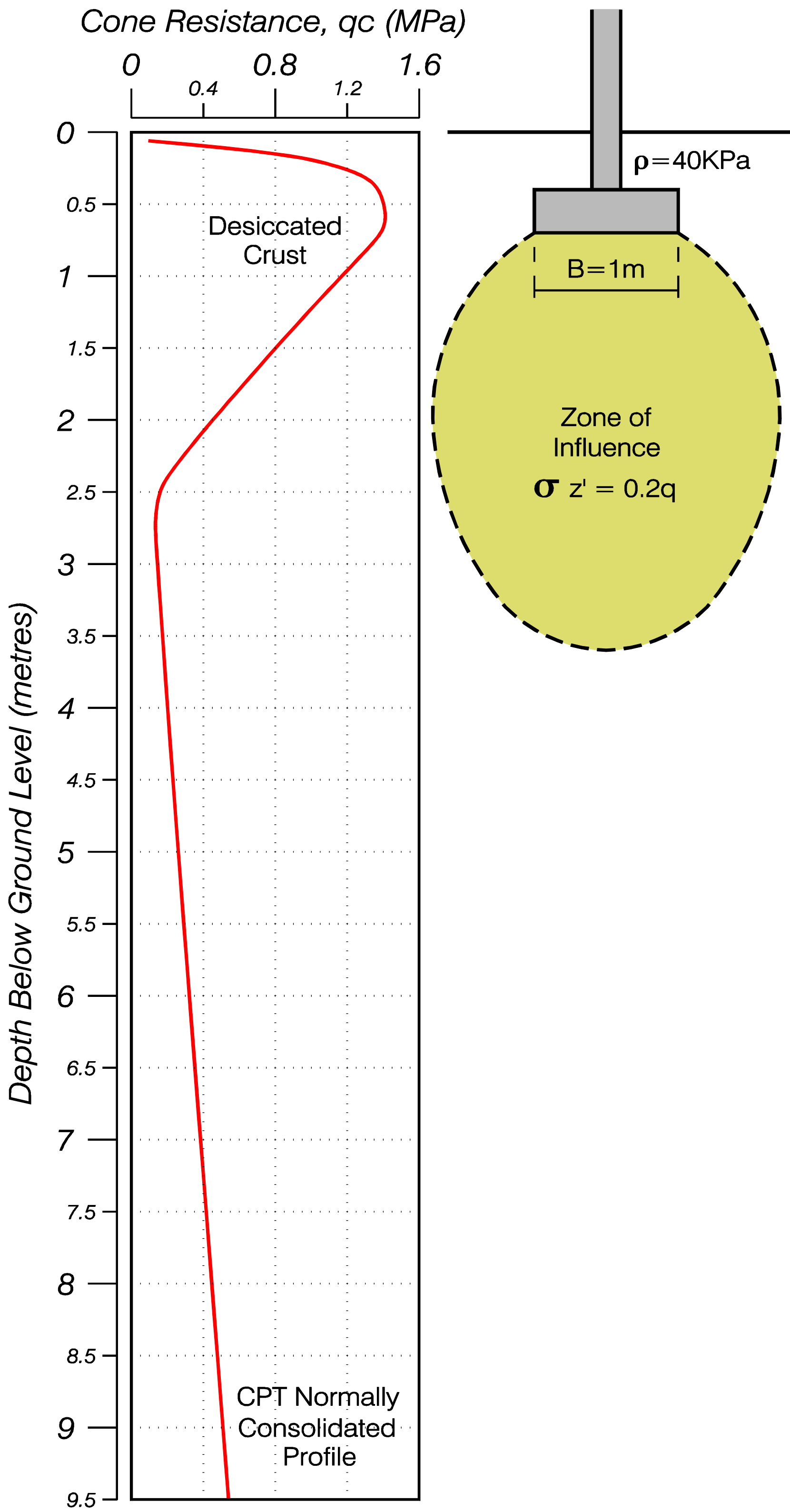
Table 1 - Correlation of cone resistance and angle of internal friction

qc= 8-12 MPa

Cone Resistance (q_c) (MPa)	Compaction of Fine Sand	SPT (N)	Relative Density Dr(%)	Angle of Internal Friction (degrees)
<2	very loose	<4	<20	<30
2-4	loose	4-10	20-40	30-35
4-12 *	medium dense *	10-30	40-60	35-40
12-20	dense	30-50	60-80	40-45
>20	very dense	>50	80-100	>45

Fugro Limited
18 Frogmore Road, Hemel Hempstead
Hertfordshire HP3 9RT
Tel: +44 1442 240781
Fax: +44 1442 258961
Email: info@fugro.co.uk www.fugro.co.uk

Fugro Limited (Scotland)
1 Queenslie Court, Summerlee Street,
Queenslie, Glasgow G33 4DB
Tel: +44 141 774 8828
Fax: +44 141 774 6112
Email: info@fsl.fugro.co.uk www.fugro.co.uk

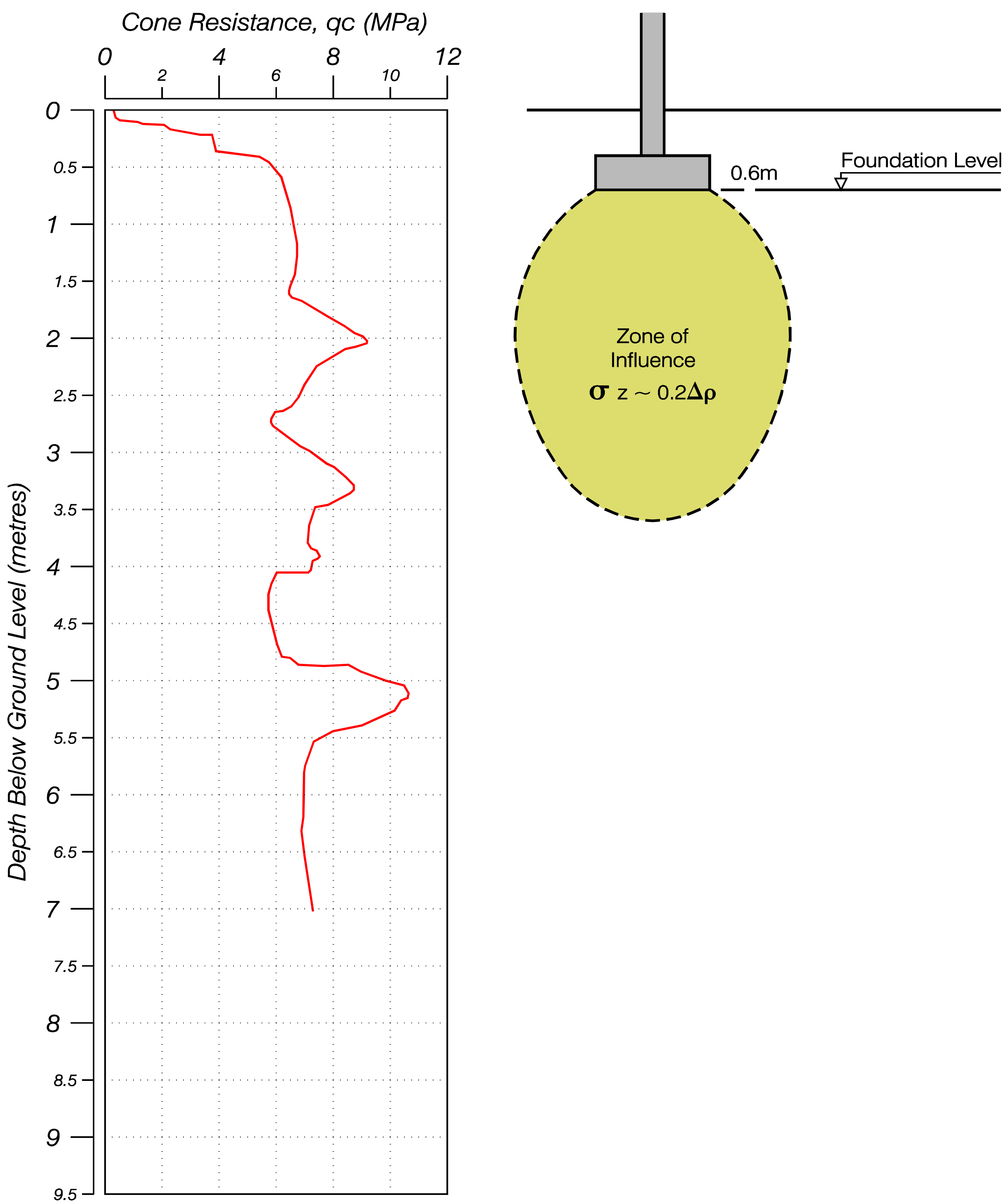


Layer	Simplified Soil Profile	Depth b.g.l. (metres)	Details	Depth Below Foundation Z (metres)	$\frac{Z}{B}$	Influence Factor If	Stress σ (KPa)	qc (MPa)	Su (KPa)	Safe Bearing Capacity (KPa)	Acceptance σ < 2 Su	mv (m^2/MN)	h (m)	s (mm)
			Ground Level											
		0.6m	Foundation Level	0.0m	0	1.0	40	1.40	77	154	ok	0.12		
1	FIRM CLAY													
		1.5m		0.9m	0.9	0.55	22	0.72	40	80	ok	0.23	0.9	4.9
2	SOFT CLAY	2.0m		1.4m	1.4	0.40	16	0.36	20	40	ok	0.46	0.5	3.3
				1.9m	1.9	0.30	12	0.14	8	16	ok	1.20	0.5	5.8
			"Weak Zone"											
		3.6m	Depth of Influence of Foundation	3.0m	3.0	0.20	8	0.20	11	22	ok	0.83	0.9	9.1
														ϵ = 23mm
3	VERY SOFT becoming SOFT CLAY						σ=If.ρ		$Su=\frac{qc}{18}$	Sbc=2.Su	ok if σ < 2 Su	$mv=\frac{1}{\alpha.qc}$ (α=6)		s=mv.σ.h
							Relevant Formulae							
		7.0m		6.4m										
4	SOFT CLAY													

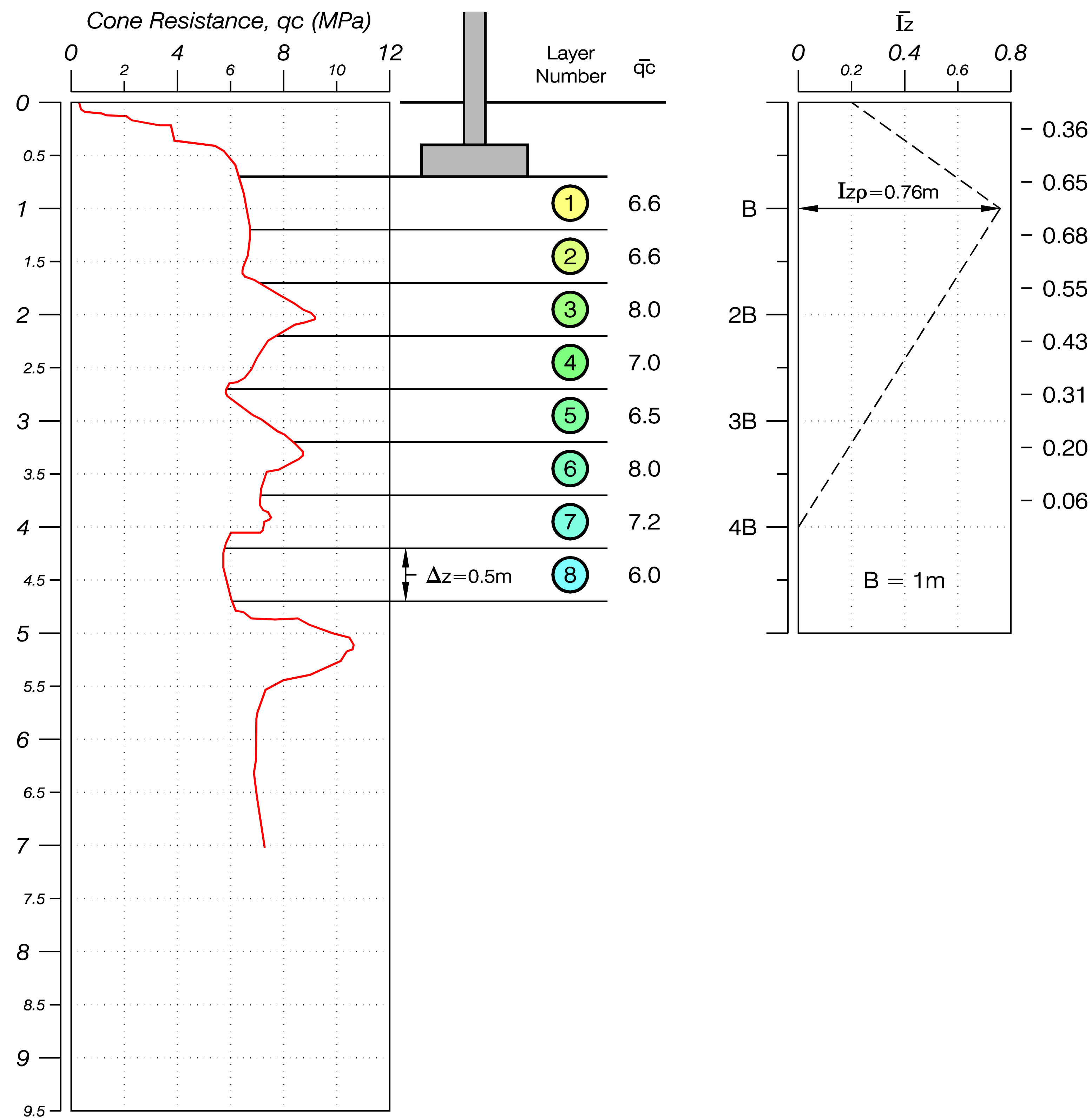
General Cone Profile and Zone of Influence of Foundation

General Calculation Format for Foundation Design in Clay

FOUNDATION SETTLEMENT ASSESSMENT IN CLAY

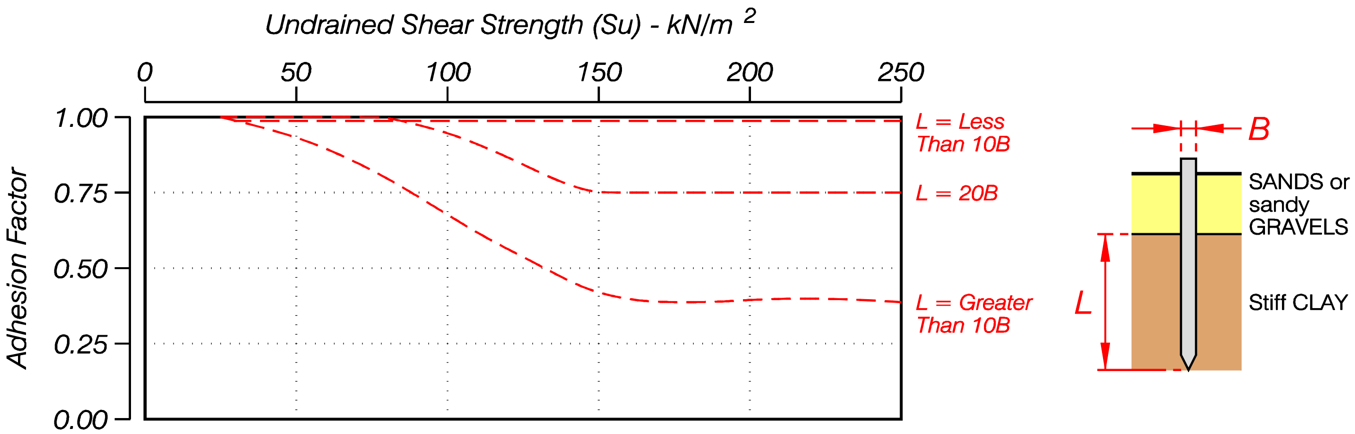


General Cone Profile and Zone of Influence of Foundation

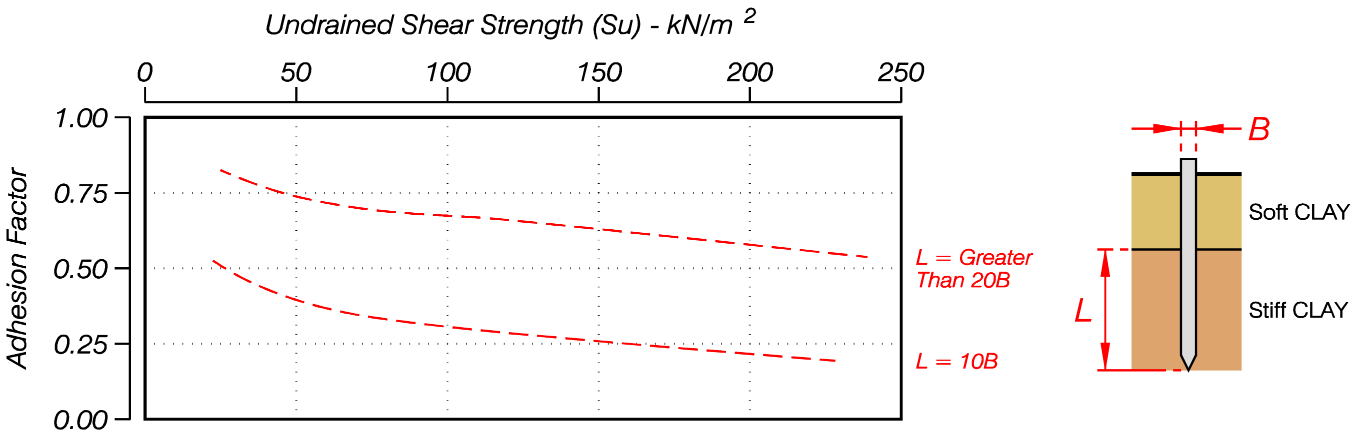


General Calculation Format for 'Schmertman' Method

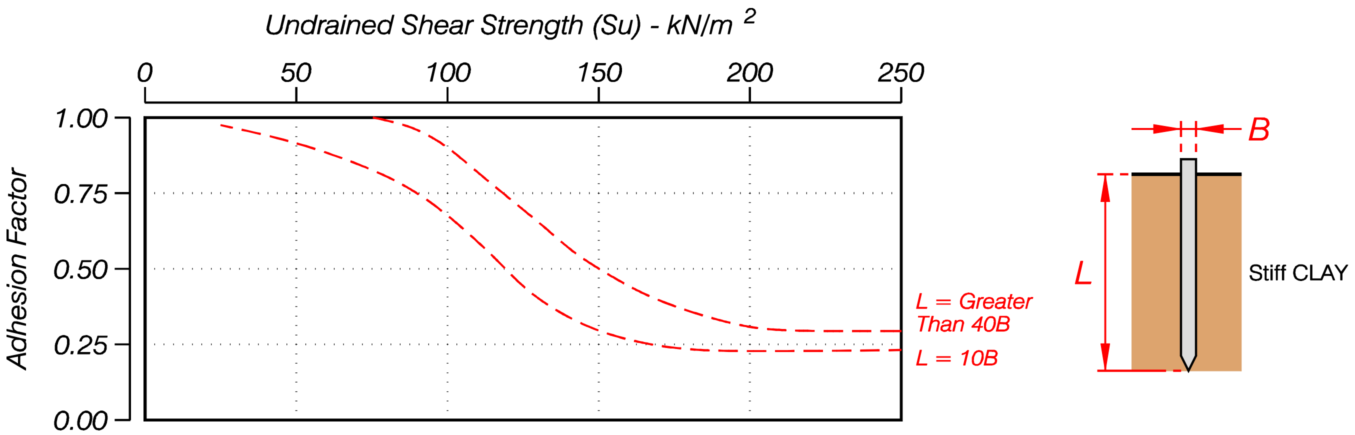
FOUNDATION SETTLEMENT ASSESSMENT IN SAND



PILES DRIVEN THROUGH OVERLYING SANDS OR SANDY GRAVELS

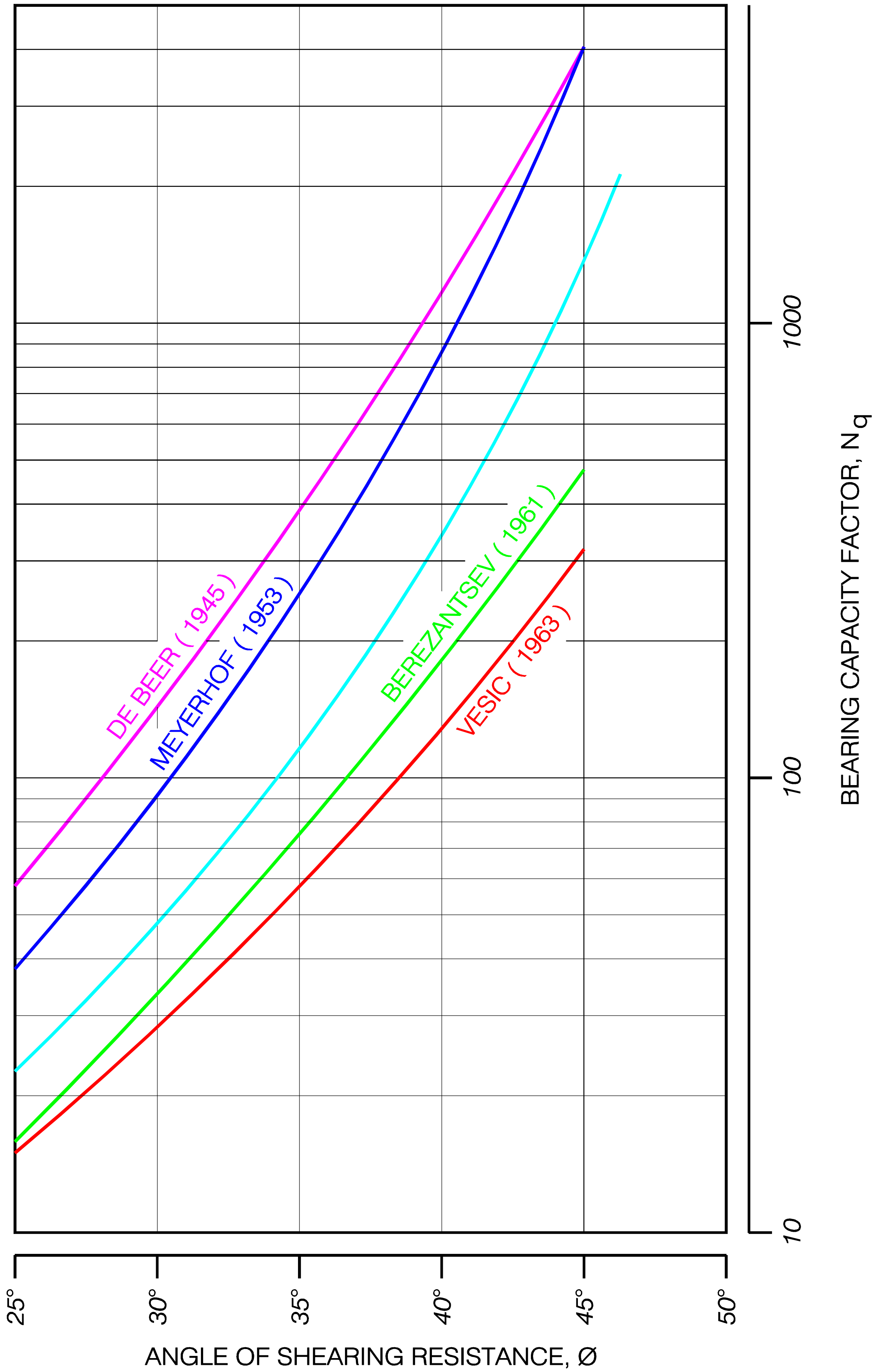


PILES DRIVEN THROUGH OVERLYING SOFT CLAY



PILES WITHOUT DIFFERENT OVERLYING STRATA

ADHESION FACTORS FOR DRIVEN PILES IN CLAY



PILE BEARING CAPACITY FACTOR

In highly compressible sands (e.g. carbonate sands or glauconitic sands), ϕ' may be significantly higher than would be derived from Figure 15. However, the presence of compressible sands can be detected from their friction ratios. If R_f (from the reference tip) exceeds about 0.5%, Figure 15 probably underestimates ϕ'. (Some carbonate sands have R_f values as high as 3% (Joustra and de Gijt, 1982), but this may not apply in cemented carbonate sands, where a reduction in R_f would be expected.) The ϕ' of cemented sands also is underestimated by Figure 15.

The method of Senneset and Janbu (1985) (set out in Section 12.2.3) requires measurement of pore pressure during the CPT. However, it can be used for estimating the angle of shearing resistance of free-draining sands without the need for pore-pressure measurements.

5.3.1 Effect of overconsolidation

The use of Figure 15 for OC sands overestimates the secant angle, ϕ', by 1 or 2°.

5.3.2 Curvature of the strength envelope

The theory underlying angles of shearing resistance presented in Figure 15 takes no account of the curvature of the strength envelope. It should be borne in mind that, at higher confining stresses, ϕ' is somewhat lower, the difference increasing with increasing relative density. Very approximately, a one-log cycle increase in confining produces a decrease in ϕ' as follows:

$D_r < 0.35$	0° to 1°
$0.35 < D_r < 0.65$	2° to 3°
$0.65 < D_r < 0.85$	3° to 5°
$0.85 < D_r$	5° to 8°

Somewhat larger reductions may occur in sands of high compressibility.

5.4 Deformability

Depending on the problem under consideration, it may be necessary to evaluate one of three moduli: the constrained modulus, M (which is equal to the reciprocal of the oedometer vertical coefficient of volume change, m_v), the Young's modulus, E, or the shear modulus, G. Furthermore, because stress-strain curves for sands are non-linear, it is necessary to fix a stress range over which the modulus is to be determined. Diagrams from which M, E and G of NC, uncemented, predominantly quartz sands can be estimated as a function of q_c and σ'_{vo} are given in Figures 16, 17, and 18, respectively. These derive from chamber tests. They may significantly underestimate the modulus values of OC sands.

PLATE 9

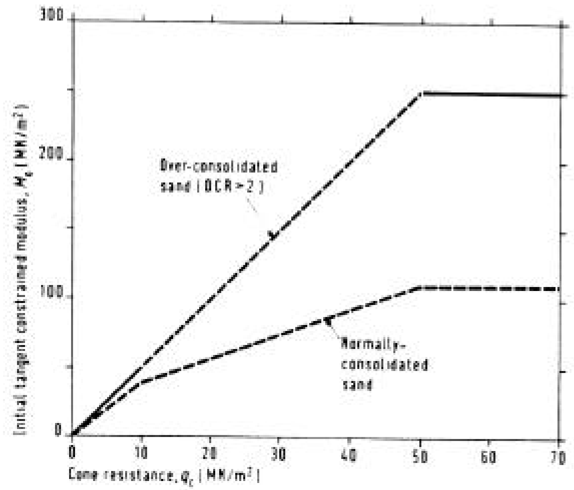


Figure 16 Initial tangent constrained modulus for normally-consolidated sands (after Lunne and Christoffersen, 1983)

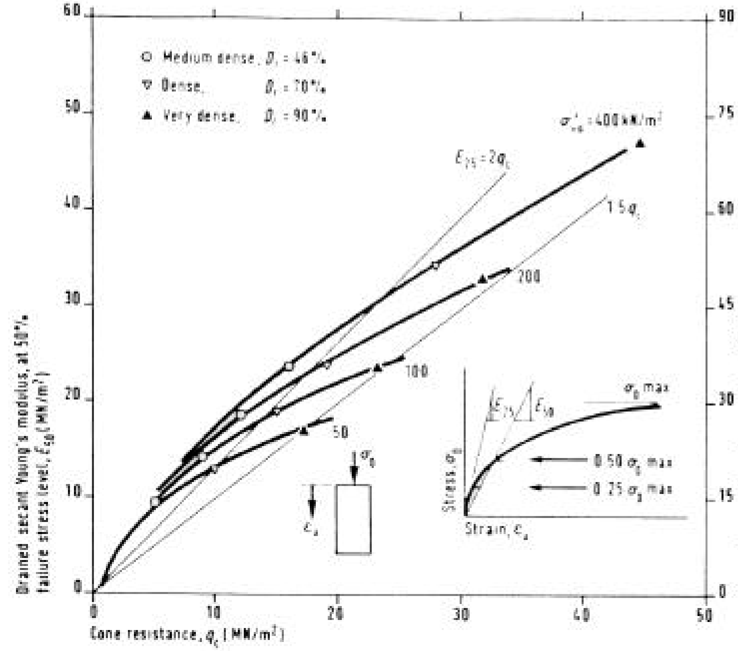


Figure 17 Secant Young's modulus values for uncemented, normally-consolidated quartz sands (after Robertson and Campanella, 1983, based on data from Baldi *et al.*, 1981)