1) Theoretical basis.

1.1) Estimating stratigraphy.

The program uses the following methods:

- ROBERTSON (1990);
- ROBERTSON (2009).

Robertson 1990

In the Robertson chart (1990) the lithological intervals are defined by the values of the normalized cone resistance Q and by the pore pressure ratio Bq. Q and Bq are given by the following expressions:

$$Q = \frac{(q_t - \sigma'_v)}{\sigma_v}$$

where:
$q_t = q_c + u_2(1\text{-}a)$
q_c = measured cone resistance;
u_2 = measured pore pressure;
a = a factor;
σ_v = total overburden stress;
σ'_v = effective overburden stress;

$$B_q = \frac{(u_2 - u_0)}{(q_t - \sigma_v)}$$

where:
u_0 = in-situ pore pressure;

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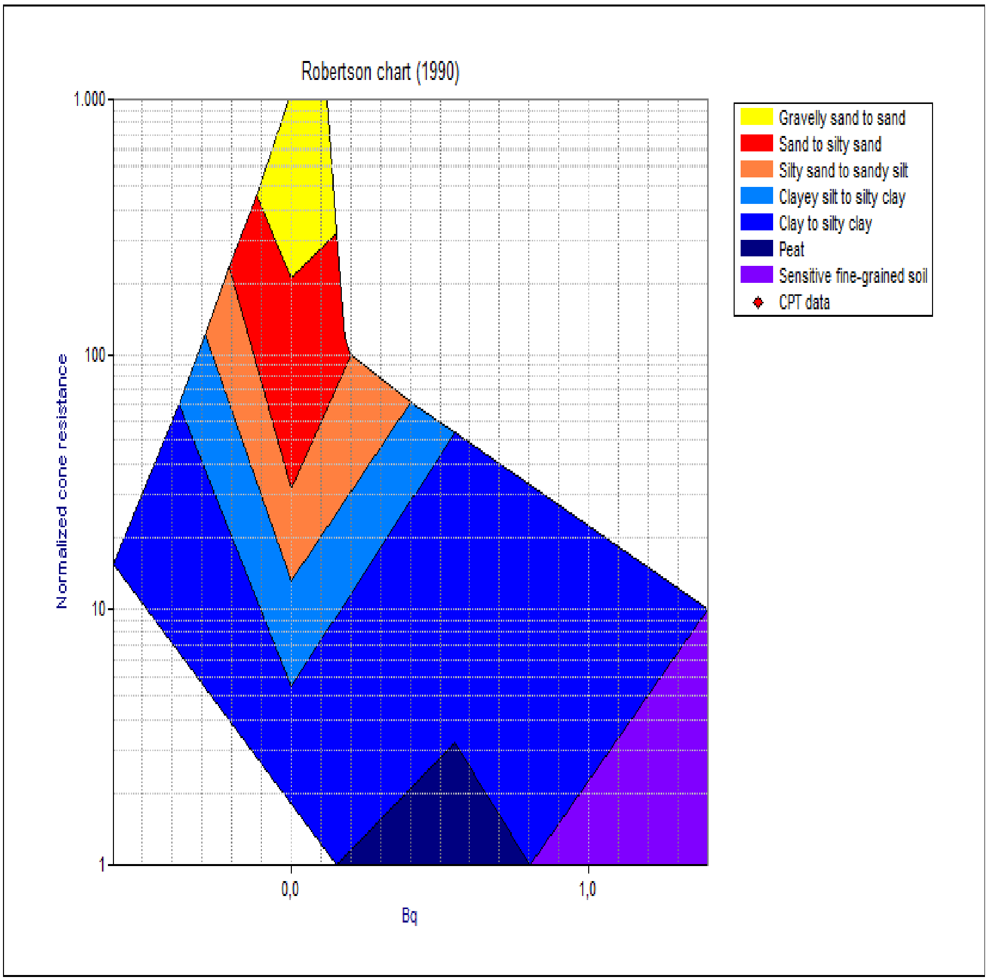


Robertson (2009)

In the Robertson chart (2009) the lithological intervals are defined by the values of the normalized cone resistance Q and by the normalized friction ratio F. Q and F are given by the following expressions:

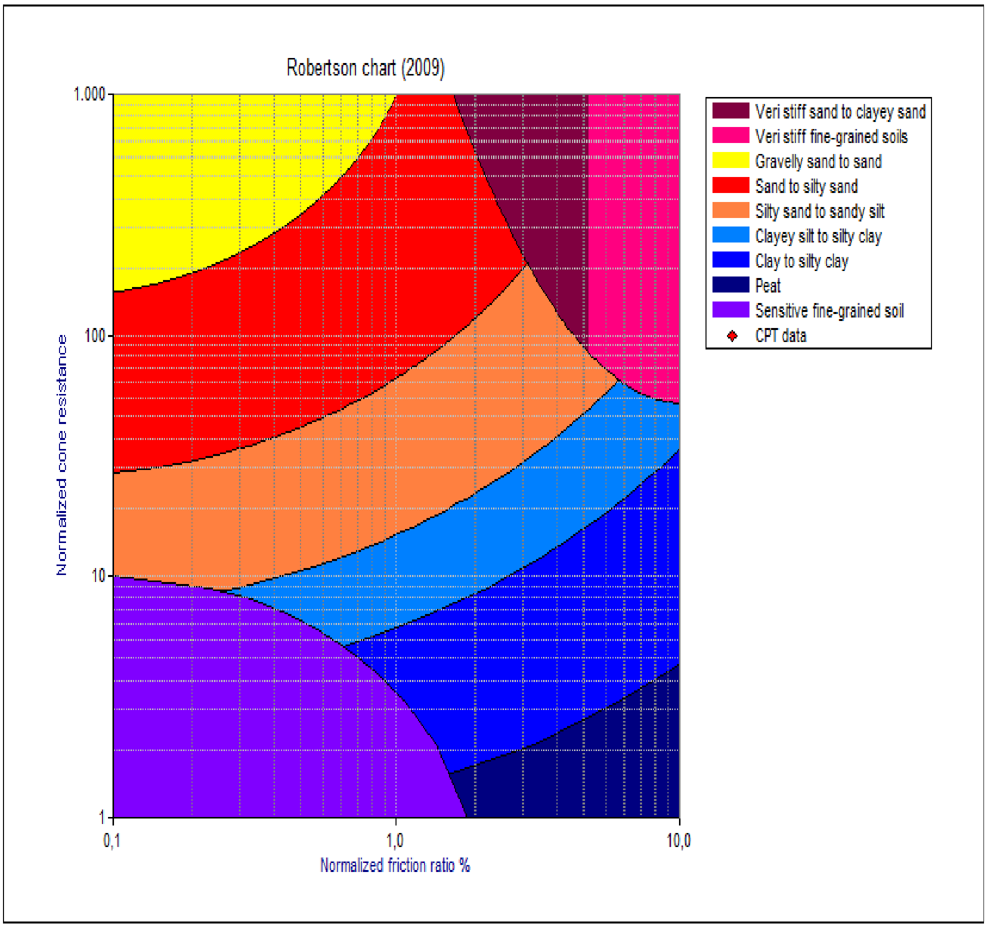
$$Q = \frac{(q_t - \sigma'_v)}{\sigma_v}$$

where:
$q_t = q_c + u_2(1\text{-}a)$
q_c = measured cone resistance;
u_2 = measured pore pressure;
a = a factor;
σ_v = total overburden stress;
σ'_v = effective overburden stress;

$$F = 100\frac{f_r}{(q_t - \sigma_v)}$$

where:
f_r = measured sleeve friction.

PROGRAM GEO – CPTu ver.4 for Windows



1.2) Estimating the geotechnical parameters.

1.2.1) Angle of shearing resistance φ.

Direct correlation methods.

a) Durgunouglu-Mitchell

The method is valid for not cemented, not consolidated sand (in case of overconsolidated sand the value has to be increased by 1-2°). The method is based on the following formula:

$$\varphi° = 14.4 + 4.8\,ln(qc) - 4.5\,ln(\sigma)$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer.

b) Meyerhof

It's based on the following relation:

$$\varphi° = 17 + 4.49\,ln(qc)$$

where qc(kg/cmq) is the cone resistance of the soil layer.
This relation is not applicable when $\varphi < 32°$ e $\varphi > 46°$. In case of overconsolidated sand the value has to be increased by 1-2°.
Meyerhof also proposed the following table to estimate approximately the angle of shearing resistance.

Normalized cone tip Resistance, q_c/p_a	Relative Density	Approximate ϕ_{tc}' (degrees)
< 20	Very loose	< 30
20 to 40	Loose	30 to 35
40 to 120	Medium	35 to 40
120 to 200	Dense	40 to 45
> 200	Very dense	> 45

c) Caquot

It's based on the following formula:

$$\varphi° = 9.8 + 4.96\,ln(qc/\sigma)$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer. In case of overconsolidated sand the value has to be increased by 1-2°.

d) Koppejan

The method is based on the following formula:

$$\varphi° = 5.8 + 5.21\,ln(qc/\sigma)$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer. In case of overconsolidated sand the value has to be increased by 1-2°.

e) De Beer

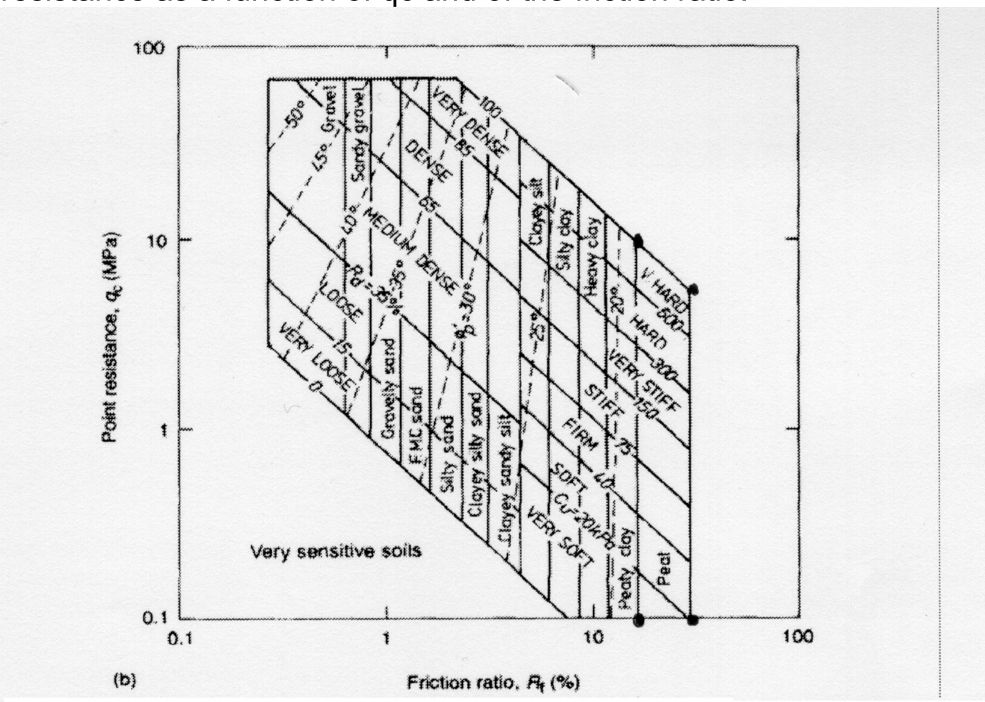
The formula is the following:

$$\varphi° = 5.9 + 4.76 \, ln(qc/\sigma)$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer. In case of overconsolidated sand the value has to be increased by 1-2°.

f) Searle

The Searle's classification allows to estimate the angle of shearing resistance as a function of qc and of the friction ratio.



g) Kulhawy and Mayne

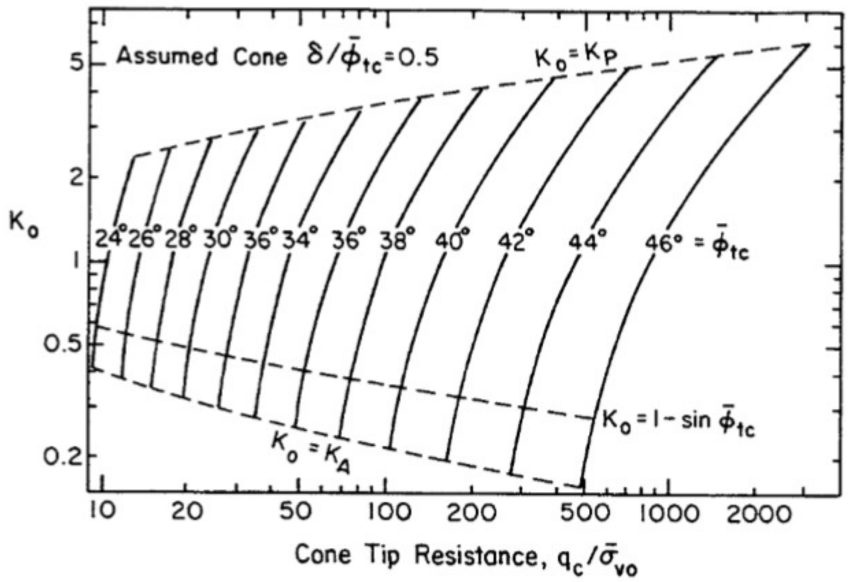
The authors suggest the following formula to approximate φ:

$$\varphi' = \tan\text{-}1[0.1+0.38\text{Log}(qt/\sigma)]$$

where qt(kg/cmq) is the average corrected cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer.

h) Marchetti

This method is based on the following chart.



i) Uzielli et al.(2013)

The formula is the following:

$$\varphi°=25.0\ (qt/(\sigma_{atm}\sigma_v)^{0.5}\ )^{0.10}$$

where qt(kg/cmq) is the average corrected cone resistance of the soil layer, σ_v(kg/cmq) is the vertical effective lithostatic pressure in the midpoint of the soil layer and σ_{atm} is the atmospheric pressure.

Indirect correlation methods.

a) Schmertmann

This method correlates the angle of shearing resistance to the relative density (D_r) of the soil layer as a function of the granulometry. It's generally valid from sand to gravel.

$\varphi = 28 + 0{,}14Dr$	Fine sand
$\varphi = 31{,}5 + 0{,}115Dr$	Medium sand
$\varphi = 34{,}5 + 0{,}10Dr$	Coarse sand
$\varphi = 38 + 0{,}08Dr$	Gravel

b) Bolton

Bolton (1986) suggests the following correlation between $\varphi_{c.v.}$ e φ_{picco}, in case of plane strain condition:

$$\varphi_{c.v.} = \varphi_{picco} - 5I_r$$

where I_r is the relative dilatancy index, varying in the interval 0÷4.
I_r is evaluated as a function of the mean effective pressure σ_n':

$\sigma_n' \leq 150$ kPa≅1,5 kg/cmq: $I_r = QD_r - 1$

$\sigma_n' > 150$ kPa≅1,5 kg/cmq: $I_r = D_r\left[Q - \ln\left(\frac{\sigma_n'}{150}\right)\right] - 1$

where D_r is the relative density, in decimal format, and Q is a parameter as a function of the mineralogical composition of the grains.

Type	Q
Quartz	5
Feldspar	5
Limestone	3
Gypsum	0,5
Q values	

The mean effective pressure is given by:

$$\sigma_n' = \frac{\bar{\sigma}_{v0} + 2\bar{\sigma}_{h0}}{3}$$

The variables σ_{v0} e σ_{h0} are respectively the vertical and horizontal effective pressure at the midpoint of the layer, taking in account that :

$$\bar{\sigma}_{h0} = K_0 \bar{\sigma}_{v0}$$

The at-rest earth pressure coefficient K_0, in normally consolidated condition, can be associated to the peak angle through the following empirical correlation(Jaki, 1967):

$$K_0 = 1 - sen\varphi_{picco}$$

1.2.2)Relative density.

a) Harman

It's valid for from fine to coarse clean sand in normally consolidated layers

$$Dr(\%) = 34.36\ln(qc/12.3\sigma^{0.7})$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer.

b) Schmertmann

It's based on the following formula:

$$Dr(\%) = -97.8 + 36.6\,ln\,qc - 26.9\,ln\,\sigma$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer.

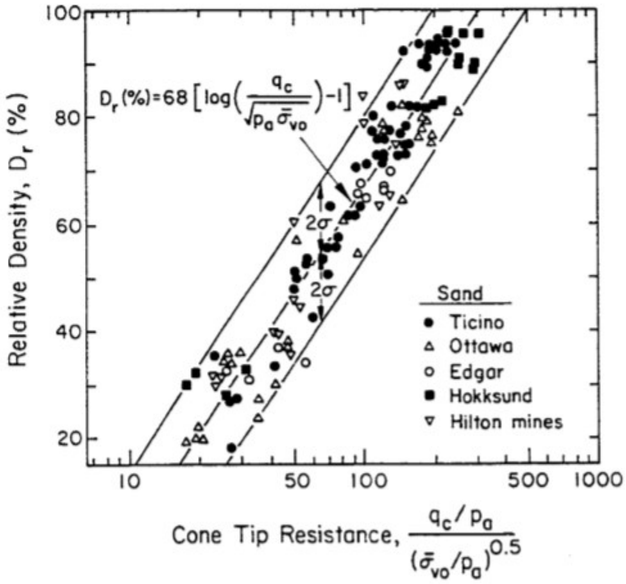
c) Meyerhof

An approximate estimate of Dr can be gotten through the following table:

Cone Tip Resistance, q_c/p_a	Relative Density	D_r (%)
< 20	Very loose	< 20
20 to 40	Loose	20 to 40
40 to 120	Medium	40 to 60
120 to 200	Dense	60 to 80
> 200	Very dense	> 80

d) Mayne

The author suggests to get Dr through the following chart.



e) Kulhawy & Mayne

The method is based on the following expression:

$$D_r = \sqrt{\left(\frac{q_c/100\text{kPa}}{305Q_c\text{OCR}^{0.18}}\right)\sqrt{\frac{100\text{kPa}}{\sigma_z^{'}}}}\text{x}100\%$$

where Qc is the compressibility factor (=0.9 to 1.1) and OCR is the overconsolidation ratio.

f) Mayne carbonate sands

The method is based on the following expression:

$$Dr\%=0.87\ qt/(\sigma_{atm}\sigma_v)^{0.5}$$

where qt(kg/cmq) is the average corrected cone resistance of the soil layer, σ_v(kg/cmq) is the vertical effective lithostatic pressure in the midpoint of the soil layer and σ_{atm} is the atmospheric pressure.

1.2.3)Young modulus

a) Schmertmann

The method is valid for normally consolidated sand in general.

$$E(kg/cmq) = 2.5\ qc$$

b) Murray

The author suggests the following correlations for several type of soils:

Soil	E
Sand (normally consolidated)	$(2 – 4)\ q_c$
Sand (overconsolidated)	$(6 – 30)\ q_c$
Clayey sand	$(3 – 6)\ q_c$
Silty sand	$(1 – 2)\ q_c$
Soft clay	$(3 – 8)\ q_c$

1.2.4)Edometric modulus (granular layers).

a) Robertson and Campanella.

This method is valid for sand in general. It's based on the following expression:

$$M(kg/cmq) = 0.03qc + 11.7\sigma + 0.79Dr\%$$

where qc(kg/cmq) is the average cone resistance of the soil layer, σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer and Dr is the relative density.

1.2.5)Shear modulus for low strain.

a) Imai and Tomauchi

This method is valid for every type of soil. It's based on the following expression:

$$G_0\ (kg/cmq) = 28qc^{0.611}$$

where qc(kg/cmq) is the average cone resistance of the soil layer.

1.2.6) Shear wave velocity.

a) Barrow & Stokoe

The authors suggest the following correlation:

$$Vs(m/s) = 50.6 + 2.1q_c$$

where q_c is in kg/cmq.

b) Mayne and Rix

This method, valid for clay, has the following expression:

$$Vs\left(\frac{m}{s}\right) = 1.75qc^{0.627}$$

1.2.7)Coefficient of permeability.

a) Piacentini and Righi

An indication of the coefficient of permeability can be obtained through the formula by Piacentini and Righi:

$$k\left(\frac{m}{s}\right) = 10^{-\left(\frac{165}{fr}+\frac{160qc}{fr^{3.5}}\right)}$$

where qc(kg/cmq) and fr are respectively the average cone resistance of the soil layer and its friction ratio (qc/fs).

1.2.8)Unit weight.

a)Mayne and Peuchen

An approximate valuation of the unit weight can be performed through the following relation:

$$\gamma = 26 - \frac{14}{\left(1 + \sqrt{(0.5Log(fs + 1))}\right)}$$

where fs is in kPa.

1.2.8)Undrained cohesion.

a) Lunne and Kelven.

This method is valid for clay in general (both n.c. and s.c.):

$$cu = \frac{qc - \sigma}{N}$$

where:
qc =average cone resistance of the layer;
N =11 for n.c. clay, 12 for s.c. clay;
σ = vertical lithostatic pressure.

b) Salgado.

This method is valid for n.c. clay in general :

$$cu = \frac{qc - \sigma}{N}$$

where:
qc =average cone resistance of the layer;
N =10;
σ = vertical lithostatic pressure.

c) Mesre et al.

The method, not usable in case of fissured clay, is based on the following formula:

$$c_u(kg/cmq) = 0{,}23\sigma'_0 \, OCR^{0,8}$$

where σ'_0 is the vertical effective pressure and OCR the overconsolidation ratio.

d) Mesre et al.

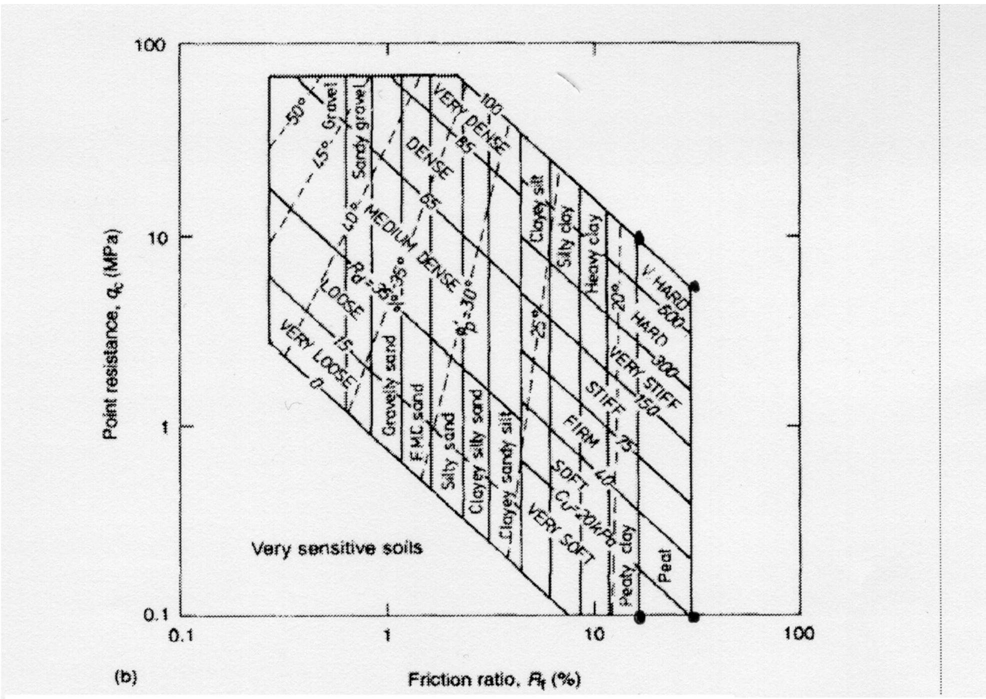
The method is valid for all the type of clay, in condition of high strain level, and it's based on the following relation:

$$c_u(kg/cmq) = 0{,}23\sigma'_0$$

where σ'_0 is the vertical effective pressure.

e) Searle

The Searle's classification allows to estimate the undrained cohesion as a function of qc and of the friction ratio.



1.2.9)Drained cohesion.

a) Mesre et al.

The method is valid for clays in general and it's based on the following relations:

$$c'(kg/cmq) = 0{,}10\sigma'_0 OCR \quad \text{(OCR}\leq 5)$$
$$c'(kg/cmq) = 0{,}062\sigma'_0 OCR \quad (5<\text{OCR}<10)$$
$$c'(kg/cmq) = 0{,}024\sigma'_0 OCR \quad \text{(OCR}\geq 10)$$

where σ'_0 is the vertical effective pressure and OCR the overconsolidation ratio.

1.2.10)Edometric modulus (cohesive layers).

a) Mitchell e Gardner

This method, valid for clay in general, is based on the following expression:

$$Ed = \alpha qc$$

where qc(kg/cmq) is the average cone resistance of the soil layer and α is a variable as a function of the type of soil:

Soils	α
CL	0.7>qc α=5
	2>qc>0.7 α=3.5
	qc>2 α=1.7
ML	2>qc α=2
	2<qc α=4.5
MH-CH	α=4
OL-OH	α=4

where qc is in Mpa.

b) Kulhawy & Mayne

This method, valid for clay in general, is based on the following expression:

$$Ed = 8.25(qc-\sigma'_0)$$

where qc(kg/cmq) is the average cone resistance of the soil layer and σ'_0 is the effective overburden presssure:

1.2.11)Overconsolidation ratio.

a) Ladd e Foot

It's based on the following formula:

$$OCR = (\frac{Cu}{\sigma KK})^{1.25}$$

where:

Cu = Undrained cohesion (Kg/cmq);
σ = Vertical effective pressure (Kg/cmq);
KK = 7-Kp, parameter as a function of depth.

Kp is calculated in the following way:

Mean depth of the layer, P(m)	Kp
P<=1	$Kp = 0.2\frac{P}{p}$
1<P<4	$Kp = (\frac{0.2}{p}) + [\frac{0.35(P-1)}{p}]$
P>4	$Kp = (\frac{0.2}{p}) + (0.35\frac{3}{p}) + [\frac{0.5(P-4)}{p}]$

where p is the sampling step of the probe.
In case of KK < 0.25, put KK = 0.25.

b) Kulhawy and Mayne

This method has the following expression:

$$OCR = 0.29\ qc/\sigma$$

where σ is the vertical effective pressure.

c) Mayne

The formula has the following expression:

$$OCR = \frac{10^{-0.7+(Log_{10}(qc)+0.22)/1.13}}{\sigma}$$

1.2.12)Compression index.

a) Schmertmann

This formula allows to calculate an estimate of the compression index:

$$Cc = 0.09 - 0.055 Log\left(\frac{2c_u}{\sigma}\right)$$

where c_u is the average undrained cohesion of the soil layer and σ(kg/cmq) is the vertical lithostatic pressure in the midpoint of the soil layer.

1.3) Bearing capacity of shallow foundations.

a) Meyerhof

Meyerhof suggested the following relations:

$$Q(Kpa) = Kd\frac{\frac{qc}{4}}{0.08} \qquad\qquad Q(Kpa) = \frac{\frac{qc}{4}}{0.05}$$

B>1.2 m B≤1.2 m

where:

Kd = 1 + 0.33(D/B), for Kd≤1.33;
D = depth of embedment of the foundation;
B = width of the foundation.

This formula has the advantage to link the bearing capacity value to the geometry of the foundation, in addition to the geotechnical characteristics of the subsoil. It has to use for mainly granular soils.
The formula gives directly the bearing capacity in respect of the ultimate limit state, without the need to add a safety factor.

b) Schmertmann

This method distinguishes between two cases as a function of the mechanical behavior of the soil layer.

Cohesive soils

$$Q(Kpa) = 2800 - 0.52^{(300-0.01qc)}$$

(strip foundation);

$$Q(Kpa) = 4800 - 0.9^{(300-0.01qc)}$$

(square foundation);

Granular soils

$$Q(Kpa) = 200 + 0.28qc$$

(strip foundation);

$Q(Kpa) = 500 + 0.34qc$

(square foundation).

where qc(kPa) is the average cone resistance of the soil layer

c) Terzaghi

This method distinguishes between two cases as a function of the mechanical behavior of the soil layer.

Cohesive soils

$$Q\lim(kg/cmq) = \gamma_1 DN_q + 0.5B\gamma_2 N_\gamma$$

where Nq=qc/0.8 and Ny=qc/0.8;

Granular soils

$$Q\lim(kg/cmq) = 2K_q\left[1 + 0.3\left(\frac{B}{L}\right)\right]$$

where K_q = qc/15, B=foundation width and L=foundation length.

1.4) Settlement of shallow foundations.

a)Terzaghi

It's based on this relation:

$$s = dH \frac{Qz}{Ed}$$

where:

dH = Thickness of the layer;
Qz = increment of the vertical pressure, due to the shallow load, in the middle point of the layer;
Ed = edometric or elastic modulus of the layer.

The calculation has to be extended to all the soil layers and the settlements summed.

$$S = \sum_{i=1}^{n} s_i \ ,$$

where n is the number of soil layers below the foundation.

b)Schmertmann

The method, used to calculate both the immediate and the secondary settlements, has the following expression:

$$S = C_1 C_2 Q \sum_{i=1}^{n} \left(\frac{Iz_i}{E_i} \cdot dH \right)$$

where:

Q = net load applied to the foundation;
C_1 = 1-0.5(σ/Q), correction factor to take in account the depth of embedment, where σ is the vertical effective lithostatic pressure at the depth of embedment

		($C_1 \geq 0.5$);
C_2	=	1 + 0.21log (T/0.1), correction factor to take in account the secondary settlement, where T is the time of calculation of the settlement in years;
σ	=	vertical effective lithostatic pressure at the depth of embedment;
n	=	number of soil layers;
dH	=	thickness of the layer;
E_i	=	elastic modulus of the i layer;
Iz_i	=	influence factor to take in account the spreading of the shallow load as a function of the depth;

1.5) Ultimate capacity of piles.

The ultimate capacity of a pile can be generally evaluated through the relation:

$$Q_{ULS} = \frac{\dfrac{Q_{point}}{F_{point}} + \dfrac{Q_{skin}}{F_{skin}}}{F_s}$$

where:

Q_{point} = pile point capacity;
Q_{skin} = skin resistance capacity;
F_{point} = safety factor of the pile point capacity;
F_{skin} = safety factor of the skin resistance capacity;
F_s = global factor of safety;

a) Meyerhof.

This method is valid for any type of pile. Q_{point} and Q_{skin} are calculated through the following relations:

$Q_{skin} = A_{skin}qc$(cohesive layers);
$Q_{skin} = 2A_{skin}qc$(granular layers);
$Q_{point} = A_{point}qc$

where:

A_{skin} = area of pile surface in mq;
A_{point} = area of pile point in mq.