Theoretical basis.

1.1) Correlation with SPT.

Since the empirical correlations available in literature between dynamic penetration test and geotechnical parameters are based on the SPT test, it needs to convert data measured with equipment having different energy of penetration (DP) to the SPT standard. That can be done through two different approaches:

- using a correction factor based on the different executive techniques: penetrometers with different characteristics have obviously different energy of penetration; to convert the number of blows obtained by a specific equipment to the equivalent number of blows of SPT, some Authors suggest to apply the following correction factor:

$$Cf = \frac{M1 \cdot H1 \cdot P11 \cdot Ap1}{M2 \cdot H2 \cdot P12 \cdot Ap2}$$

where:

M2	=	SPT hammer weight (63.5 kg);
H2	=	SPT falling height (75 cm);
Pl2	=	SPT sampling step (15 cm);
Ap2	=	SPT area of the sampler or of the cone (20.4 cmq);
M1	=	DP hammer weight;
H1	=	DP falling height;
Pl1	=	DP sampling step;
Ap1	=	DP cone area.

The number of blows to use in the calculation of the geotechnical parameters will be gotten by the following formula:

$$Nspt = CfNscpt$$

The Cf coefficient can be seen as an efficiency ratio of the penetration test, on energy basis, in respect to the SPT standard. To the SPT standard is normally associated a mean efficiency ratio of 60%. Thus Cf can be expressed in the following way:

$$Cf = \frac{E_{scpt}\%}{E_{spt}\%} = \frac{E_{scpt}\%}{60}$$

where E_{spt} is the efficiency of penetration of the SPT standard. However it recommends using the efficiency ratio given by the constructor of the equipment.

- correction on the base of the lithology: the ratio between number of blows SPT and DP tends towards 1 in coarse granulometries, while it's higher than 1 in fine granulometries; so it's possible to suggest the following corrections:

Correction	Lithology
NSPT = 1 x NDP	Gravel and sandy gravel
NSPT = 1.25 x NDP	Sand and gravel with fine
NSPT = 1.5 x NDP	Silty and clayey sand
NSPT = 2 x NDP	Silt
NSPT = 2.5 x NDP	Sandy and silty clay
NSPT = 3 x NDP	Clay

They are empirical corrections to use carefully and, if it's possible, it's recommended to calibrate them on local basis.

1.2) Lithology and dynamic resistance.

1.2.1)Lithology

A rough evaluation of the lithology can be performed on the base of the ratio between the number of blows of the point and of the sleeve, if present:

Ratio Npoint/Nsleeve	Lithology
< 0,25	Clay
0,25 - 0,40	Silty or sandy clay
0,40 - 0,70	Silt
0,70 - 2,25	Clayey or silty sand
2,25 – 4	Sand or gravel with fine
> 4	Gravel or Sand+Gravel

1.2.2)Dynamic resistance

The dynamic resistance of the penetrometer is calculated through the following formula :

$$Rd(Kg/cmq) = \frac{P^2 H}{ApRf(P + Pa + Pt)}$$

dove:

P (kg) = hammer weight;
H (cm) = falling height;
Ap (cmq) = area of the point;
Rf (cm) = sampling step/Nspt;
Pa (kg) = total weight of the rods;
Pt (kg) = weight of the drive head.

1.3) Geotechnical parameters.

1.3.1) Parameters of the granular soils

1.3.1.1) Peak angle of shearing resistance φ.

Direct correlaction methods

a) Road Bridge Specification

The method is valid for silty-clayey sand and has its best conditions of applicability in the case of depth higher than 8-10 m in dry soils and higher than 15 m in saturated soil (σ > 0.15-0.20 MPa).
The method is based on the following formula:

$$\varphi = \sqrt{15Nspt} + 15$$

where Nspt is the mean value of the number of blows in the layer.

b) Japanese National Railway

The method is valid from medium-coarse to gravelly sand and has its best conditions of applicability in the case of depth higher than 8-10 m in dry soils and higher than 15 m in saturated soil (σ > 0.15-0.20 MPa).
The method is based on the following formula:

$$\varphi = 0{,}3Nspt + 27$$

where Nspt is the mean value of the number of blows in the layer.

c) De Mello

The De Mello method is valid for any type of sand and for any depth (apart the first 2 m from the ground surface).
It's not attendible with values of φ higher than 38°.

The method is based on the following formula:

$$\varphi = 19 - 3{,}8\sigma + 8{,}73 Log\left(N_{spt}\right)$$

where σ is the effective lithostatic pressure at the midpoint of the layer in kg/cmq and Nspt is the mean value of the number of blows in the layer.

d) Owasaki & Iwasaki

The method is valid from medium-coarse to gravelly sand and has its best conditions of applicability in the case of depth higher than 8-10 m in dry soils and higher than 15 m in saturated soil (σ > 0.15-0.20 MPa).
The method is based on the following formula:

$$\varphi = \sqrt{20 Nspt} + 15$$

where Nspt is the mean value of the number of blows in the layer.

e) Sowers

The Sowers method (1961) is valid for any type of sand has its best conditions of applicability in the case of depth lower than 4 m in dry soil and lower than 7 m in saturated soil (σ < 0.05-0.08 MPa)
This is the formula:

$$\varphi = 28 + 0{,}28 N_{spt}$$

f) Malcev

The Malcev method is valid for any type of sand and for any depth (apart the first 2 m from the ground surface).
It's not attendible with values of φ higher than 38°.

$$\varphi = 20 - 5Log(\sigma) + 3{,}73Log(N_{spt})$$

where σ is the effective lithostatic pressure at the midpoint of the layer in kg/cmq and Nspt is the mean value of the number of blows in the layer.

g) Peck-Hanson & Thornburn

The Peck-Hanson & Thornburn method is valid for any type of sand and has its best conditions of applicability in the case of depth lower than 5 m in dry soils and lower than 8 m in saturated soil (σ < 0.08-0.10 MPa).

$$\varphi = 27{,}2 + 0{,}28N_{spt}$$

h) Meyerhof

The Meyerhof method (1965) is valid for any type of sand and has its best conditions of applicability in the case of depth lower than 5 m (formula 1) or 3 m (formula 2) in dry soils and lower than 8 m (formula 1) or 5 m(formula 2) in saturated soil.

(for.1) $\varphi = 29{,}47 + 0{,}46N_{spt} - 0{,}004N^2{}_{spt}$ (< 5% of silt)

(for.2) $\varphi = 23{,}7 + 0{,}57N_{spt} - 0{,}006N^2{}_{spt}$ (>5% of silt)

i) Hatanaka e Uchida

This method proposes a correlation between φ and the parameter N1(normalized penetration resistance as a function of the effective lithostatic pressure of 1 kg/cmq).
N1 is calculated by the Liao and Whitman relation (1986):

$$N1 = N_{spt}\left(\frac{1}{\sigma_{v0}'}\right)^{0.5}$$

where σ_{v0}' is the effective lithostatic pressure at the midpoint of the layer in kg/cmq.
The peak angle of shearing resistance is given by the formula:

$$\varphi = \sqrt{20N1} + 20$$

l) Terzaghi

Terzaghi method (1953) is based on the following table:

Nspt	φ°
<4	28
4-10	28-30
10-30	30-36
30-50	36-41
>50	41

It's possible to interpolate the values of the table to obtain intermediate values.

m) Kulhawy e Mayne

The method is valid for any type of sand and has the following expression:

$$\varphi = \arctan\left[\frac{N_{spt}}{(12{,}2 + 20{,}3\sigma')^{0{,}34}}\right]$$

where σ is the effective lithostatic pressure at the midpoint of the layer in kg/cmq.

n) Wolff

This method proposes a correlation between φ and the parameter N1(normalized penetration resistance as a function of the effective lithostatic pressure of 1 kg/cmq).

N1 is calculated by the Liao and Whitman relation (1986):

$$N1 = N_{spt}\left(\frac{1}{\sigma_{v0}'}\right)^{0.5}$$

where σ_{v0}' is the effective lithostatic pressure at the midpoint of the layer in kg/cmq.
The peak angle of shearing resistance is given by the formula:

$$\varphi = 27,1 + 0,3N1 - 0,00054N1^2$$

Indirect correlation methods

a) Schmertmann

This method correlates φ to the relative density of the layer as a function of its granulometric composition. It's valid for sand and 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 $\varphi_{c.v.}$ and 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':

σ_n'≤150 kPa≅1,5 kg/cmq: $I_r = QD_r - 1$

σ_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 into 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):

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

1.3.1.2)Relative density.

The relative density can be evaluated through correlations applicable only in the case of sandy layers. In presence of gravelly soils the calculated values tend to overestimate the relative density.

a) Gibbs & Holtz

The Gibbs & Holtz method (1957) is valid for clean sand for any value of effective pressure inside normally consolidated soil layers.

$$Dr(\%) = 21\sqrt{\frac{N_{spt}}{\sigma + 0{,}7}}$$

where σ is the effective lithostatic pressure at the midpoint of the layer in kg/cmq and Nspt is the mean value of the number of blows in the layer.

b) Schultze & Mezembach

The Schultze & Mezembach method (1961) is valid for clean sand for any value of effective pressure inside normally consolidated soil layers.

$$\ln(Dr\%) = 0{,}478\ln(N_{spt}) - 0{,}262\ln(\sigma) + 2{,}84$$

c) Skempton

The method is valid for clean sand for any value of effective pressure inside normally consolidated soil layers.

$$Dr(\%) = 100\sqrt{\left(\frac{N_{spt}\sqrt{\frac{98}{\sigma}}}{32 + 0.288\sigma}\right)}$$

d) Skempton 1986

This method proposes a correlation between Dr and the parameter N1(normalized penetration resistance as a function of the effective lithostatic pressure of 1 kg/cmq).
N1 is calculated by the Liao and Whitman relation (1986):

$$N1 = N_{spt}\left(\frac{1}{\sigma_{v0}'}\right)^{0.5}$$

where σ_{v0}' is the effective lithostatic pressure at the midpoint of the layer.
The relative density is calculated by the following formula:

$$Dr\% = 100\sqrt{\frac{N_1}{60}}$$

e) Cubrinovski

The method is valid for clean sand for any value of effective pressure inside normally consolidated soil layers.
The relative density is calculated by the following formula:

$$Dr(\%) = 100\sqrt{\left[\frac{N_{spt}\left(0,23 + \frac{0,06}{D_{50}}\right)^{1,7}}{9}\sqrt{\frac{98}{\sigma'}}\right]}$$

where:

σ' = effective lithostatic pressure in kPa;
N_{spt} = mean value of the number of blows;
D_{50} = mean grain size

f) Terzaghi

Terzaghi method (1953) is based on the following table:

Nspt	Dr%
<4	15
4-10	15-35
10-30	35-65
30-50	65-85
>50	85

It's possible to interpolate the values of the table to obtain intermediate values.

1.3.1.3)Young modulus.

a) Tornaghi et al.

The method is valid for sand+gravel or clean sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of E as a function of the depth.
It's based on the following expression:

$$E(MPa) = B\sqrt{Nspt}$$

where B is a constant equal to 7 Mpa.

b) Schmertmann

The method is valid for any type of sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of E as a function of the depth.
It's based on the following expression:

$$E(kg/cmq) = 2BNspt$$

where B is a constant as a function of the lithology:

B	Lithology
4	Fine sand
6	Medium sand
10	Coarse sand

c) Stroud

The method is based on the following expression:

$$E(MPa) = \alpha Nspt$$

where α is a parameter as a function of Nspt:

$$\alpha = -0.00107Nspt^2 + 0.136Nspt + 1.503 .$$

d) D'Appolonia et Alii.

The D'Appolonia method is valid for sand+gravel and overconsolidated sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of E as a function of the depth.

$$E(kg/cmq) = 7{,}71 N_{spt} + 191 \text{ (Sand + Gravel)}$$
$$E(kg/cmq) = 10{,}63 N_{spt} + 375 \text{ (SC sand)}$$

e) Schultze e Menzebach.

The Schultze e Menzebach method is valid for saturated sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of E as a function of the depth.

$$E(kg/cmq) = 5{,}27 N_{spt} + 76$$

f) Webb.

The Webb method is valid for saturated sand or silty-clayey sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of E as a function of the depth.

$$E(kg/cmq) = 4{,}87 N_{spt} + 73 \text{ (saturated sand)}$$
$$E(kg/cmq) = 3{,}22 N_{spt} + 16 \text{ (silty-clayey sand)}$$

g) Kulhawy & Mayne.

The Kulhawy & Mayne method is valid for clean sand or clayey sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of E as a function of the depth.

$$E(kg/cmq) = 5N_{spt} \text{ (clayey sand)}$$
$$E(kg/cmq) = 10N_{spt} \text{ (NC clean sand)}$$
$$E(kg/cmq) = 15N_{spt} \text{ (SC clean sand)}$$

1.3.1.4)Edometric modulus.

a) Farrent.

The Farrent method is valid for any type of sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of M as a function of the depth.

$$M(kg/cmq) = 7{,}1N_{spt}$$

b) Menzebach & Malcev.

The Menzebach & Malcev is valid for any type of sand. The formula doesn't consider the effect of the effective pressure, which involves a decrease of M as a function of the depth.

$$M(kg/cmq) = 3{,}54N_{spt} + 38 \text{ (Fine sand)}$$
$$M(kg/cmq) = 4{,}46N_{spt} + 38 \text{ (Medium sand)}$$
$$M(kg/cmq) = 10{,}46N_{spt} + 38 \text{ (Sand+gravel)}$$
$$M(kg/cmq) = 11{,}84N_{spt} + 38 \text{ (Gravelly sand)}$$

1.3.1.5)Low strain shear modulus.

a) Ohsaki & Iwasaki

The Ohsaki & Iwasaki method, valid for clean sand or clayey-silty sand, is based on the following formula:

$$G_0\,(t\ /\ mq) = aNspt^{b}$$

where a and b are constants as a function of the granulometric composition:

a	b	Granulometria
650	0.94	Clean sand
1182	0.76	Silty-clayey sand

b) Crespellani & Vannucchi

The Crespellani & Vannucchi method, valid for any type of sand, is based on the following expression:

$$G_0(t/mq) = 794Nspt^{0,611}$$

1.3.1.6)Shear wave velocity.

a) Ohta & Goto

The authors suggests the following formula:

$$Vs(m/s) = C_s N_{spt}{}^{0.171} z^{0.199} F_a F_g$$

where:

C_s =	Empirical constant = 67.3
z (m)=	Depth of the midpoint of the layer
F_a =	Factor as a function of the geological age of the layer F_a =1 for holocene soils F_a =1.3 for pleistocene soils
F_g =	Factor as function of the granulometric composition F_g = 1.45 for gravels F_g = 1.15 for gravelly sands F_g = 1.14 for coarse sands

	F_g = 1.07 for medium sands F_g = 1.09 for fine sands

1.3.1.6)τ/σ ratio.

The liquefaction resistance of a sand can be evaluated through the empirical expression by Seed and Idriss.

$$\tau / \sigma = \frac{N_1}{90}$$

where:

$$N_1 = \left[1 - 1{,}25 Log_{10}\left(\sigma_v '\right)\right] N_{spt}$$

1.3.2) Parameters of the cohesive soils

1.3.2.1)Undrained cohesion.

α) Terzaghi & Peck

The method is usable in case of medium plasticity clay and is based on the following relation:

$$c_u (kg / cmq) = 0{,}067 N_{spt}$$

β) DM-7 (Design Manual for Soil Mechanichs)

The method, valid for clay in general, is based on the following relations:

$c_u (kg / cmq) = 0{,}038 N_{spt}$ (low plasticity clay)

$$c_u(kg/cmq) = 0{,}074N_{spt}$$ (medium plasticity clay)

$$c_u(kg/cmq) = 0{,}125N_{spt}$$ (high plasticity clay)

c) Sanglerat

The method, valid for low and medium plasticity clay, is based on the following relations:

$$c_u(kg/cmq) = 0{,}125N_{spt}$$ (medium plasticity)

$$c_u(kg/cmq) = 0{,}100N_{spt}$$ (silty clay)

$$c_u(kg/cmq) = 0{,}067N_{spt}$$ (silty-sandy clay)

d) Shioi - Fukui

The method, valid for high and medium plasticity clay, is based on the following relations:

$$c_u(kg/cmq) = 0{,}025N_{spt}$$ (medium plasticity clay)

$$c_u(kg/cmq) = 0{,}05N_{spt}$$ (high plasticity clay)

e) Hara et al.

The method is valid in case of medium plasticity clay and it's based on the following formula:

$$c_u(kg/cmq) = 0{,}29N_{spt}^{\ 0{,}72}$$

f) 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.

g) 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.

1.3.2.2)Edometric modulus.

a) Stroud e Butler

It's valid for low and medium plasticity clay:

$$E_d(kg/cmq) = 5N_{spt} \text{ (medium plasticity clay)}$$
$$E_d(kg/cmq) = 6N_{spt} \text{ (low plasticity clay)}$$

1.3.2.3)Overconsolidation ratio.

a) Ladd e Foot

It's based on the following formula:

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

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, KK = 0.25.

b) Kulhawy e Mayne

The overconsolidation ratio (OCR) can be estimated through the following formula (Kulhawy & Mayne):

$$OCR = 0,58\frac{N_{spt}}{\sigma'_0}$$

1.3.2.4)Low strain shear modulus .

a) Ohsaki & Iwasaki

It's based on the following formula:

$$G_0\,(t\,/\,mq) = aNspt^{b}$$

where a = 1400 and b = 0.78.
The method is valid for cohesive layers in general.

1.3.2.5)Shear wave velocity .

a) Ohta & Goto

The authors suggests the following formula:

$$Vs(m/s) = C_s N_{spt}^{0.171} z^{0.199} F_a F_g$$

dove:

C_s =	Empirical constant = 67.3
z (m)=	Depth of the midpoint of the layer
F_a =	Factor as a function of the geological age of the layer F_a =1 for holocene soils F_a =1.3 for pleistocene soils
F_g =	Factor as function of the granulometric composition F_g = 1.00 for clay

1.3.2.6)Drained cohesion.

a) Mesre et al.

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

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

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

1.4) Bearing capacity of shallow foundations.

a) Meyerhof

Meyerhof suggested the following relations:

$$Qamm(KPa) = \left(\frac{B+0.3}{B}\right)^2 \frac{Nspt}{0.06} Kd \ , \quad \text{B>1.2 m}$$

$$Qamm(KPa) = Kd \frac{Nspt}{0.04} \ , \quad \text{B} \leq \text{1.2 m}$$

where:

Kd = 1 + 0.33(D/B), for Kd≤1.33;
Nspt = mean value of the number of blows measured in the layer;
D = depth of embedment of the foundation;
B = width of the foundation;

This formula has the advantage to associate 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) Dutch formula

The dutch formula is based on the following relation:

$$Q\lim(Kg/cmq) = \frac{P^2 H}{20 ApRf(P + Pa + Pt)}$$

where:

P (kg) = hammer weight;
H (cm) = falling height;
Ap (cmq) = area of the point;
Rf (cm) = ratio between the sampling step and the number of blows per step (30/Nspt);
Pa (kg) = total weight of the rods
Pt (kg) = weight of the drive head.

The Dutch formula doesn't allow to correlate the bearing capacity to the geometry of foundation. Thus it has to be used carefully and only for rough estimates.

c) Parry

The method of Parry is based on the following formula:

$$Qamm(KPa) = \frac{30Nspt}{Fs}$$

where: Fs = safety factor.

1.5) 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.

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

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:

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

dove:

Q	=	net load applied to the foundation;
C_1	=	1-0.5(σ/Q), correction factor to take into account of 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 into 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;

1.6) Ultimate capacity of piles.

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

$$Q_{SLU}(t) = \frac{\dfrac{Q_{punta}}{F_{punta}} + \dfrac{Q_{laterale}}{F_{laterale}}}{F_s}$$

where:

Q_{punta} = pile point capacity;
$Q_{laterale}$ = skin resistance capacity;
P_{palo} = weight of the pile;
F_{punta} = safety factor of the pile point capacity;
$F_{laterale}$ = safety factor of the skin resistance capacity;
F_s = global factor of safety;

a) Meyerhof.

It's valid only in case of precast piles:

$$Q_{laterale}(t) = 0{,}2 A_{lat} N_{spt}$$
$$Q_{punta}(t) = 40 A_{base} N_{spt}$$

where:

A_{lat} = area of the effective pile surface in mq;
A_{base} = area of the effective pile point in mq.