PROGRAM GEO - GBMuri ver.1 per Windows

Index

1. Theoretical bases

1.1 Earth pressure coefficients

ACTIVE EARTH PRESSURE COEFFICIENT.

The active earth pressure coefficiente can be considered, on a first approximation, as the minimum ratio between the stress acting on the horizontal plane (retaining of the soil wedge upslope) and the one on the vertical plane (weight of the overlieyng soil and possible loads on the soil ground), applied to a soil element in condition of limit plastic equilibrium.

$$Ka = Phmin / Pv.$$

The active earth pressure is actived when soil undergoes a decompression (a decreasing of the horizontal pressure to which does not correspond an equal variation of the vertical pressure) with strains of the order of 0.2-0.3%.
One can determine a plane along which Ka reaches its minimum value. This plane represents a potential failure surface along which the soil wedge isolated by the failure plane itself will push on the retaining wall placed downslope.

PASSIVE EARTH PRESSURE COEFFICIENT.

Pressing along the horizontal plane the soil and keeping inalterate the vertical pressure, the value of P_h rises up to reach a maximum value. This condition is named state of upper plastic equilibrium limit or passive state and it can be reached for high strains of the soil only (2%-4%). Passive state is normally generated in the downslope soil of a retaining wall due to the displacement it undergoes for the earth pressure upslope and it has the effect to contrast the shifting of the wall itself.

Among the available methods to assess K_a e K_p there are:

- Rankine;
- Muller-Breslau;
- Mononobe e Okabe;
- Caquot Kerisel;
- Trial wedge.

1.1.1Rankine's method

Setting φ(°) as the value of the angle of shear resistance of the soil, the active earth pressure coefficient is given by:

$$K_a = tg^2(45° - φ/2);$$

The potential failure surface is planar and start form the heel of the wall with a slope of 45°+ φ/2.
The passive earth pressure coefficient instead can be evaluated by the formula:

$$K_p = tg^2(45° + φ/2).$$

Such a method needs, to be used, both the horizontal and vertical planes be principal stress planes. By practise it happens when:

- the stem of the retaining wall is vertical;
- there is no friction along the contact surface between the stem and the soil (angle of shear resistence stem-soil=0).

As far as this last point, it has to be considered that the presence of shear stress along the stem take to a significatly decreasing of the active earth pressure. Neglet this stress drives to more conservative values of Ka and of the total pressure .

1.1.2 Muller-Breslau's method

In the Muller-Breslau's method the condition that the stresses acting on the horizontal and vertical planes are principal stress planes is not imposed. The resultant of the total earth pressure is consequently inclined by a specific angle equal to the angle of shear resistance between the stem and the soil.
Set:

- β = slope of the stem of the wall with respect to the vertical plane;
- ρ = slope of the soil failure plane;
- δ = wall-soil angle of shear resistance, often set equal to arctg[2/3 x tg(φ)];
- ε = slope of the topographic profile upslope;
- φ = soil angle of shear resistance;

the active earth pressure is given by:

$$Ka=\cos^2(\varphi-\beta)/[\cos^2\beta \cos(\delta+\beta)(1 + \sqrt{Rp})^2]$$

with
$Rp=sen(\varphi+\delta)sen(\varphi-\varepsilon)/[\cos(\delta+\beta)\cos(\varepsilon-\beta)]$;

the active earth pressure is instead given by:

$$Kp=\cos^2(\varphi+\beta-\theta)/[\cos\theta \cos^2\beta \cos(\delta-\beta+\theta)(1 - \sqrt{Rp})^2]$$

with
$Rp=sen(\varphi+\delta)sen(\varphi+\varepsilon-\theta)/[\cos(\delta-\beta+\theta)\cos(\varepsilon-\beta)]$;

This method is applicable to the most of practical cases, with an error contained within 5% with respect to more elaborated methods, provided the condition $\delta \leq \varphi / 3$ be verified.

1.1.3 Monobe & Okabe's method

The Mononobe & Okabe's method is similar to the Muller-Breslau's one. They fundamentally differ for the way the seismic effect is considered.
With:

- β = slope of the stem of the wall with respect to the vertical;
- ρ = slope of the soil failure plane;
- δ = angle of shear resistance between soil and stem, often set equal to arctg[2/3 x tg(φ)];
- ε = slope of the topographic profile upslope the wall;
- φ = angle of shear resistance of the soil;
- θ = angle related to the seismic force, set equal to 0 in static condition.

the active earth pressure coefficient is given by:

$$Ka=\cos^2(\varphi-\beta-\theta)/[\cos\theta \cos^2\beta \cos(\delta+\beta+\theta)(1 + \sqrt{Rp})^2]$$

where
$Rp=sen(\varphi+\delta)sen(\varphi-\varepsilon-\theta)/[\cos(\delta+\beta+\theta)\cos(\varepsilon-\beta)]$;

the passive earth pressure coefficient is instead given by:

$$Kp=\cos^2(\varphi+\beta-\theta)/[\cos\theta \cos^2\beta \cos(\delta-\beta+\theta)(1 - \sqrt{Rp})^2]$$

with
$Rp=sen(\varphi+\delta)sen(\varphi+\varepsilon-\theta)/[\cos(\delta-\beta+\theta)\cos(\varepsilon-\beta)]$;

1.1.4 Caquot-Kerisel's method

In case where be $\delta>\varphi/3$ the approximation gotten applying the Muller-Breslau's method is no more neglectable.

The potential soil failure plane has to be approximate to a logarithmic spiral arc and not to a plane surface.
The active earth pressure coefficient by Caquot-Kerisel can be assess through the following relationship:

$$Kc = p \times K0;$$

with
$p=a\ b;$
$K0=10^{(w\ f)};$
in cui:
$a=[\cos(\beta'-\varphi)^2/\cos(\beta'+\delta)];$
$b=\{1/[1+\sqrt{(sen(\varphi+\delta)sen(\varphi-\varepsilon)/\cos(\beta'+\delta)\cos(\beta'-\varepsilon))}]\}^2;$
$w=-Log[(1-0.9l^2-0.1l^4)(1-0.3l^3)];$
$f=\sqrt{(sen\ \varphi)}[2-(tg^2\varepsilon + tg^2\delta)/(2\ tg^2\varphi)];$
$l=(\beta'-\beta)/(\beta'+\beta+\pi-2\varphi);$
$b0=(m+\varepsilon-r)/2;$
$r=arcsen(sen\ \varepsilon/sen\ \varphi)$
$m=2arctg\{[cotg\ \delta-\sqrt{(cotg^2\delta-cotg^2\varphi)}]/(1+cosec\varphi)\};$
$\beta'=90°-\beta;$

Per la stima del coefficiente di spinta passiva si adotta invece la formula già vista per il modello di Muller-Breslau.

1.1.5 Method of the trial wedge

In case of layer or topographic profiles having an irregular shape, a more realistic approach is given by the method of the trial wedge.
By a practical point of view, it consists in the making of a set of potential failure planes, with increasing slope, strating from the heel of the wall and instersecting the topographic profile upslope.
Each wedge isolated in this way, of weight W, produces a reaction R having a slope φ with respect to the normal to the failure surface.

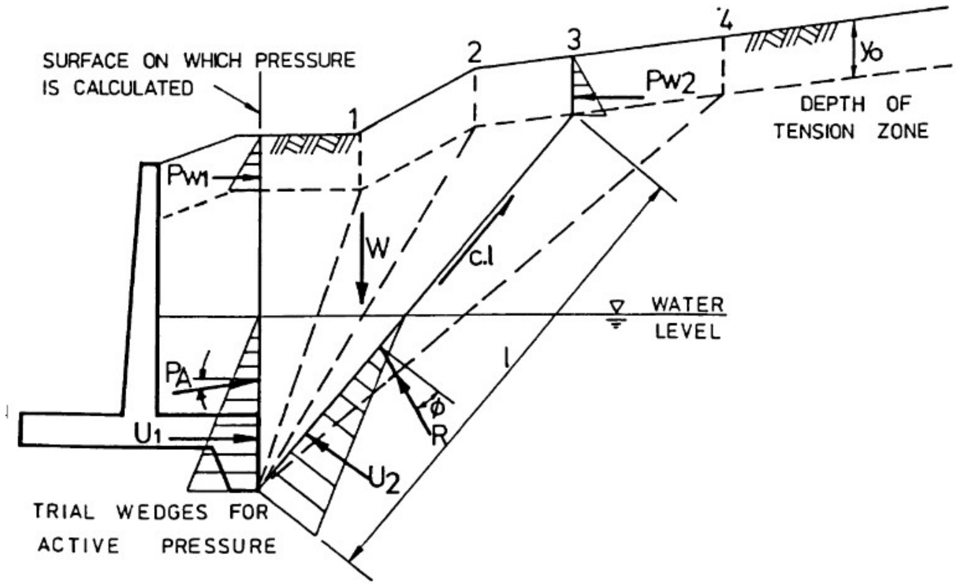
In case of presence of cohesion have to be added the contribution due to the adesion wall-soil C_w(Coulomb condition) and to the cohesion acting along the potential failure plane C_s.

$$C_w = H\ ca \quad C_s = d\ c$$

where

H =	Lenght of the stem
ca =	Wall-soil adesion
d =	Length of the potential failure plane
c =	Soil cohesion

In case of water table, the pressures due to water, U1 and U2, have to be considered. The vectorial sum of the forces acting on the wall gives the value of the total earth pressure. The calculation has to be repeated for every wedge and the maximum value has to be assumed as value of reference.



1.2 Assessment of the active and passive earth pressures

On first approximation one can assess the horizontal active earth pressure, Ka known, by the formula:

$$Ph = Pv\ Ka$$

In case of homogenous soil, without cohesion and no water table, on which acts the gravity force only, one can write:

$$Ph = \gamma\ z\ Ka;$$

where

γ = soil unit weight;

z = depth from the ground.

The product γ z is the pressure of the lithological column at the depth z.
By integration on the total height of the wall, one can get:

$$(a)\ Sa = 0.5\ H^2\ \gamma\ Ka;$$

with Sa = active earth pressure.
The resultant of the earth pressure, from the plane of embedment of the wall, is equal to:

$$l = H/3.$$

To the relationshiop to assess Sa, others components have to be added, if any, due to:

- multilayer soil;
- water table;
- cohesion;
- external loads
- seismic forces;
- irregular topographic profile;

1.2.1 Multilayer soil

Consider, as example, a soil with three layers having different geotechnical parameters.
The calculation of the active earth pressure has to be execute in the following way:

- the relationship (a) is applied to each layer, substituting to H and γ the thickness and unit weight values of the layer and the correspondent computed Ka relative to the angle of shear resistance; the resultant of the active earth pressure is given by:

$$ls = Hs/3 + \Sigma(da\ H1\ a\ Hs\text{-}1)\ H;$$

consequently, in the case of a three layer soil, the resultant of the layer 3, the shallower, is given by:

$$l3 = H3/3 + H2 + H1.$$

- the contribution as load of each layer with respect to the lower ones is calculated; so the total contribution to the active pressure given by the layer 1, the deepest, will be:

Sa1'=0.5 $H1^2$Ka3 γ3(contribute of the layer 1)
Sa1"=(γ2H2+γ3H3)H3 Ka3(contributions of the layers 2 and 3 as load on the layer 1);

the resultan is given by:

$$l1=[(H1/3)Sa1'+(H1/2)Sa1'']/(Sa1'+Sa1'').$$

In the same way, for the layers 2 and 3:

Sa2'=0.5 $H2^2$Ka2 γ2(contribution of the layer 2)
Sa2"=(γ3xH3)H2 Ka2(contributions layer 3 as load on layer 2);

Sa3'=0.5 $H3^2$Ka3 γ3(contribution of the layer 3)
Sa3"= 0;

$$l2=\{[(H2/3)+H1]Sa2'+[(H2/2)+H1]Sa2''\}/(Sa2'+ Sa2'');$$

$$l3=\{[(H3/3)+H2+H1]Sa1'+[(H1/2)+H2+H1]Sa1''\}/ (Sa1'+Sa1'').$$

The total active earth pressure is given by:

$$Sa=(Sa1'+Sa1'')l1+(Sa2'+Sa2'')l2+(Sa3'+Sa3'')l3/(Sa1'+Sa1''+Sa2'+Sa2''+Sa3'+Sa3'').$$

1.2.2 Water table

In case of water table, the relationship (a), for the submerged layers, changes as follow:

$$Saw=0.5\ \gamma'\ Ka\ Hw^2;$$

with
γ'=submerged unit weight;
Hw=height of the water table with respect to the base of the wall;

having a resultant equal to:

$$law = Hw/3.$$

For the layers above the water table H has to be replaced with H-Hw. The resultant of the earth pressure of the layers above the water table is given by:

$$l = Hw + (H-Hw)/3.$$

The contribution due to the hydraulic pressure has to be considered too:

$$Sw = 0.5\ Hw^2\ \gamma w,$$

with a resultant:

$$lw = Hw/3,$$

The contribution due to the load of the dry soil volume on the submrged one is given by:

$$Sa' = (H-Hw)\gamma\ Hw\ Ka,$$

with γ = unit weight above the water table, with a resultant:

$$la' = Hw/2.$$

1.2.3 Cohesive soils

Presence of cohesion drives to a reduction of the active pressure.
At a given depth z, supposing for simplicity a homogeneous soil and no water table and loads, total active pressure is:

$$(b)Ph = \gamma \ z \ Ka - 2 \ c \ \sqrt{Ka},$$

with c = soil cohesion.

The first term (γ z Ka) represents the earth pressure as a function of the depth in a soil not cohesive; the second term is the constant contribution due to the cohesion.
By integration along the wall height, one has:

$$Sa = 0.5 \ \gamma \ H^2 \ Ka - 2c \ H \ \sqrt{Ka},$$

with a resultant:

$$la = H/3.$$

Near the ground surface (z close to 0), the second term in (b) get greater, in absolute value, than the first one and the active earth pressure gets negative.
So the shallower layer is undergone by tractive force and break up, making a tension crack. Depth of this crack is given setting Ph=0 in (b) and solving with respect to z:

$$Zc = 2c \ / \ (\gamma \ \sqrt{Ka}).$$

To the calculation of the earth pressure the contribuition of the layer crossed by the tension crack has to be considered equal to 0. Supposing a triangular diagram of pressure:

$$Sc' = 0.5(Zc\ 2\ c\ \sqrt{Ka}.)$$

Then replacing to Zc its expression:

$$Sc' = 2c^2/\gamma.$$

Sc' represents a compensative term of the negative earth pressure in the shallower layer undergone to tension.
The (a) expression has to be modified as follows:

$$Sa = 0.5\ \gamma\ H^2 Ka - 2c\ H\ \sqrt{Ka} + Sc',$$

with a resultant:

$$la = (H - Zc)/3.$$

Moreover, with a lack of efficient drainage of shallow water upslope, the tension cracks could filled up by water, driving to an increasing of the earth pressure, computable as follows:

$$Scw = 0.5\ Zc^2,$$

with a resultant of the pressure equal to:

$$lcw = (H - Zc) + Zc/3.$$

1.2.4 External loads

They are considered three kinds of external loads acting on the ground upslope:

- uniform loads;

- point loads;
- strip loads.

1.2.4.1 Uniform load

They are loads of elevate areal exstension, starting from the stem of the wall and of uniform intensity.
Setting q=load pressure, the contribution given to the active earth pressure is:

$$Su = q\ H\ Ka\ [\mathrm{sen}\ \beta\ /\ \mathrm{sen}\ (\beta+\varepsilon)],$$

with a resultant:

$$lu = 0.5\ H.$$

It is not correct, as proposed by several Authors, to transform, as an alternative, an uniform load into equivalent height of soil, rewriting (a) in the follows way:

$$Sa = 0.5\ \gamma\ Ka\ (H + Heq),$$

with $Heq = q\ [\mathrm{sen}\ \beta\ /\ \mathrm{sen}\ (\beta+\varepsilon)]/\gamma$.

1.2.4.2 Point loads

A point load is a load with a very small areal extension. The contribution to the active earth pressure can be obtained by the elastic solution proposed by Boussinesq:

$$\sigma r = (Q/2\pi)\{(3r^2z/R^5)-[(1-2\mu)/(R^2+zR)]\}.$$

where:

σr = radial component of the pressure at the depth z of a small volume with coordinates x,y with respect to the point of application of the load;
Q = modulus of the load;
$r = \sqrt{(x^2 + y^2)}$;
$R = \sqrt{(r^2 + z^2)}$;
μ = Poisson's ratio (usually set equal to 0.35).
Through σr the horizontal pressure at the depth z is given by:

$$\sigma h = \sigma r(x/r).$$

By numerical integration with a fixed step, the total contribution to the active earth pressure due to the point load is calculated, with a resultant:

$$(31)\ lsc = \Sigma PiHi/\Sigma Pi;$$

with
Hi=height from the wall base;
Pi=pressure due to the load at the height Hi.

1.2.4.3 Strip loads

They are loads of significant areal extension, placed parallel to the length of the wall, covering a limited portion of the slope behind the wall. The load pressure is considered uniform along the loaded area.
The contribution to the active earth pressure is calculated as in case of point loads. By practise, the loaded area is subdivised in a higher number of small point loads and then the contributions are summed.
In the same way one can proceeded to compute the resultant of the earth pressure.

1.2.4.4 Seismic forces

To take in account the effect of the seismic forces on the active earth pressure, Ka has to be recomputed, setting:

$$\theta = \text{arctang}\,(kh/(1\text{-}kv)).$$

where kh and kv are respectively the horizontal and vertical seismic coefficients, linked to the horizontal seismic acceleration by the expressions:

$$k_h = \beta a_{\max} \text{ e } k_v = 0.5 k_h$$

where amax is the horizontla peak ground acceleration. Parameter β is often set to 0.38, in case of deformable wall, and to 1 otherwise.
The increasing of seismic pressure is given by the difference between the earth pressure in seismic and static conditions.

$$\Delta S = Sa' - Sa.$$

The resultant is:

$$l\Delta S = (2/3)H.$$

1.2.4.5 Slope with irregular profile

In case of slope behind the wall having an irregular profile, one can proceedes setting the slope horizontal ($\varepsilon = 0$) and considering the soil above as a set of strip loads of small size (e.g. 0.1 m). The load pressure is calculated as:

$$Q\ (t/m) = \Delta l \times h \times \gamma;$$

where:
Δl = width of the loade area (e.g. 0.1 m);

h = average height of the soil column;
γ = soil unit weight.

1.2.5 Passive earth pressure

To calculate the passive earth pressure, the same procedures seen in case of the active earth pressure has to be used.
Obviously the active pressure coefficient Ka has to be replaced by the passive one Kp. Moreover, the factor $2c\ H\ \sqrt{Kp}$, link to the cohesion, if any, has to be taken with the sign + and summed to the other pressure components.

1.3 Tiebacks.

The pull-out resistance of a tieback T can be calculated through the formulas by Schneebeli and Bustamante Doix.

Schneebeli

In case of granular soil (φ>0) the formula is the following:

$$T_l = \pi D_p L tg\left(45 - \frac{\varphi}{2}\right) \operatorname{sen}\varphi \frac{1+e^{2\pi tg\varphi}}{2} \gamma Z \; ;$$

where:

D_p =borehole diameter;
L =length of the bond;
Z =depth of the median point of the bond;
γ =unit weight of the soil above the bond.

In case of cohesive soil in undrained condition (φ=0) one can instead use the expression:

$$T_l = \pi D_p Lc$$

c= undrained cohesion of the soil along the anchored bulge.

Pull-out resistance is given by the ratio between Tl and a safety factor, usually chosen equal to 2.5:

$$T = \frac{T_l}{2{,}5}$$

Bustamante Doix

The expression is the following:

$$T_l = \pi \alpha D_p L q_s \; ;$$

dove:

D_p =borehole diameter;
L =length of the bond;
α =factor which measures increasing of the bond diameter.

qs =lateral friction or adhesion along the bond.

The α coefficient is function of the prevalent lithology along the bond and of the grouting method. One can assess it by the following table:

Lithology	Coefficient α	
	repeated grouting	*simple grouting*
Gravel	1.8	1.3-1.4
Sandy gravel	1.6-1.8	1.2-1.4
Gravely sand	1.5-1.6	1.2-1.3
Clean sand	1.4-1.5	1.1-1.2
Silty sand	1.4-1.5	1.1-1.2
Silt	1.4-1.6	1.1-1.2
Clay	1.8-2.0	1.2
Marl and sandstone weathered and/or fractured	1.8	1.1-1.2

The qs factor can be obtained by these formulas:

simple grouting :

$q_s(MPa) = 0{,}01(Dr - 50) + 0{,}05$ granular soil (Dr=relative density)

$q_s(MPa) = 0{,}006(c - 10) + 0{,}1$ cohesive soil (c=cohesion in t/mq)

repeated grouting :

$q_s(MPa) = 0{,}01(Dr - 50) + 0{,}1$ granular soil(Dr=relative density)

$q_s(MPa) = 0{,}008(c - 10) + 0{,}18$ cohesive soil(c=cohesion in t/mq)

Pull-out resistance is given then by the ratio between T1 and a safety factor, usually posed equal to 2.5

$$T = \frac{T_l}{2{,}5}$$

Placing and sizing tiebacks have to be executed taking in account that:

- bond has to be placed at a depth higher than the potential sliding plane to perform its stabilizing action;

- optimum inclination of the tieback can be assess by the formula:

$$i_{optimum} = arctg(\tan \varphi / Fs)$$

where:
φ=angle of shear resistance of the soil;
Fs=safety factor.

In case of permanent tiebacks it has to take in account the decreasing of the pull-out resistance as a function of time. Then T has to be divided to an additional safety factor (β coefficient) usually set equal to 1.5.

1.4 Stability of retaining walls.

It has to be assessed the wall stability with respect to:

- forward sliding;
- overturning;
- bearing capacity failure.

1.3.1 **Forward sliding.**

It has to be performed, comparing the horizontal forces which push the retaining wall downslope (unstabilizing forces) and the ones which contrast the sliding (stabilizing forces).

1.3.1.1 Stabilizing forces

The following variables are defined:

Na = Lm(Wmuro+Wterra+Svert+Cvert) lcr /Bmuro;

with
Wterra=weight of the wedge of soil laying on the wall foundation upslope;
lcr=width of the key, if any (lcr=0 otherwise);

Ta = Lm(Soriz+Coriz) lcr /Bmuro;

Wa = Lm scr γ dcr/2 (weight of the triangular prism of soil between the external edge of the wall foundation and the key, if any);
with
scr=width of the key, if any;
γ =unit weight of the soil below the foundation;
dcr=distance of the key, if any, from the downslope edge of the foundation.

Nb = Lm(Wmuro+Wterra+Svert+Cvert)- Na;
Tb = Lm(Soriz+Coriz) - Nb;

The total stabilizing force is given by:

$$Fstab=Pl + Na\ fa + [(Nb + Wa)\cos\theta + Tb\ sen\ \theta]fa' + C\ Bef;$$

with
θ=arctg (scr / dcr)=slope of the line joining the downslope edge of the foundation to the bottom of the key;
fa=angle of sher resistance wall-soil;
fa'=angle of shear resistance along the sliding plane, which can assume the following values:
a) fa'=tg φ (φ = angle of shear resistance of the soil below the foundation) when there is a key;
b) fa'=angle of shear resistance wall-soil, if there is no key;

C=cohesion acting along the sliding plane, which can assume the following values:
a) C=cohesion of the soil, when there is a key;
b) C=(2/3) cohesion of soil, if there is no key;

Bef=length of the potential sliding plane of the wall, which can assume the following values:
a) Bef=[(scr / sen θ) + lcr] + (Bmuro- (lcr+dcr), if there is a key;
b) Bef=Bmuro if there is no key.

1.3.1.2 Unstabilizing forces

The total unstabilizing force is given by:

$$Finstab=Ta + Tb\cos\theta - (Nb + Wa)\ sen\ \theta;$$

The degree of stability of the retaining wall is given by the ratio:

$$Fsic = Fstab / Finstab,$$

defined as safety factor to the forward sliding.

1.3.2 **Overturning**

It has to be performed, comparing the unstabilizing moments and the stabilizing ones acting on the retaining wall, referred to the toe.

1.3.2.1 Stabilizing moments

a) Component due to the wall weight:

Ms1=Lm Wmuro Xb.

b) Component due to the vertical active earth pressure.

Ms2=Lm Svert(Bmuro - Ys cos β).

c) Component due to the weight of the soil laying on the wall foundation:

Ms3=Lm Wterra Bmuro.

d)Component due to the group of piles, if any.

Ms4=Pl(2/3)Lpalo.

e)Component due to vertical loads acting on the top of the wall, if any.

Ms5=Lm Cvert Xb.

f)Component due to moments acting on the top of the wall, if any.

Ms6=Mest.

1.3.2.2 Unstabilizing moments

a)Component due to the horizontal active earth pressure.

$$Mr1=Lm\ Soriz\ Ys;$$

b)Component due to horizontal loads acting on the top of the wall, if any.

$$Mr2=Lm\ Coriz\ Hmuro.$$

c)Component due to moments acting on the top of the wall, if any.

$$Mr3=Mest.$$

The degree of stability of the retaining wall is given by the ratio:

$$Fsic = \Sigma Mstabilizzanti / \Sigma Mribaltanti.$$

Defined as safety factor to overturning.

1.3.3 Bearing capacity failure

It has to be performed, comparing the bearing capacity of the wall foundation to the sum of the vertical loads.
In case of shallow foundation:

$$Fsic=Beb \cos \alpha\ Qamm /(Wmuro+Wterra+Svert+Cvert);$$

with
Fsic = safety factor;
Beb = width of the wall foundation corrected to take in account the eccentricity due to the vertical loads;
Qamm=bearing capacity of the shallow foundation;
α=inclination of the foundation.

In case of foundation on a pile group:

$$Fsic= Pamm /(Wmuro+Wterra+Svert+Cvert);$$

with
Pamm=capacity of the pile group.

1.3.4 **Maximum and minimum pressures on the wall base.**

To compute the maximum and minimum pressures applied to the soil below the wall base, the following expressions are used:

$$Pmax=(Nvertcos\ \alpha/B)(1+6e\cos\alpha/B)$$
$$Pmin=(Nvertcos\ \alpha/B)(1-6\ e\cos\alpha/B);$$

if $e < B/6$
with
Nvert=resultant of the vertical loads:
Nvert=(Wmuro+Wterra+Svert+Cvert);
B=width of the base;
α=slope of the base;
e=load eccentricity=B/2-r
where r=(ΣMstabilizing - ΣMunstabilizing)/Nvert;

if $e > B/6$

$$Pmax=(2Nvert)/[3(B/2-e)]\ Pmin=0;$$

if $e = B/6$

$$Pmax=(2N)/H\ Pmin=0;$$

with H =wall height.

1.3.5 **Piping**

In case of seepage flow on the bottom of the excavation, a verification of the piping risk has to be performed.
A simplified method to quantify the safety factor of the wall to piping is the Terzaghi's, based on the following expression:

$$Fs = \frac{D\gamma'}{h_a \gamma_a}$$

dove:

D	= depth of embedment of the wall;
γ'	= submerged unit weight of the soil;
h_a	= pore pressure at the depth D, which can be set equal to 0.5H, where H=depth of the excavation;
γ_a	= water unit weight;

The verification is satisfied if the safety factor is greater than 1.

1.3.6 **Internal stability**

The internal stability verifications have the aim to verify the capacity of the material compousing the wall to tolerate the normal and tangential forces due to the weight of the wall itself and to the forces and the moments acting from outside.
The maximum strength to the normal stress can be directly gained by test or through empirical formulas, like the following:

$$\sigma_{mat}\ (kPa) = 50\ \gamma_{mat} - 294.3$$

where γ_{mat} is the unit weight of the filling material given in kN/mc.
This value has to be confronted with the maximum value of the normal stress applied, choosing between the highest of the stress acting, respectively, on the downslope and upslope side.

$$\sigma_{vi} = \frac{N}{B_{base}} \cdot \left(1 + \frac{6 \cdot e}{B_{base}}\right)$$

$$\sigma_{mi} = \frac{N}{B_{base}} \cdot \left(1 - \frac{6 \cdot e}{B_{base}}\right)$$

Where N is the normal force, due to the sum of the wall weight and of external forces, B_{base} is the width of the checked section, whereas e is given by:

$$e = \frac{B_{base}}{2} - u$$

with

$$u = \frac{M_{stab} - M_{instab}}{N}$$

where M_{stab} and M_{instab} are respectively the stabilizing and unstabilizing moments acting on the downslope edge of the checked section.

The maximum strength to the tangential stress can be gained by empirical formulas, like the following:

$$\tau_{mat}\,(kPa) = \sigma_n \, tg\varphi_{mat} + c_g$$

where:

σ_n = normal stress applied by the wall weight and the external normal forces;

φ_{mat} = angle of shear strength of the filling material, assessed by the following empirical formula

$$\varphi_{mat}\,° = 2.55\,\gamma_{mat} - 10$$

c_g = virtual cohesion due to the wiring mesh, assessed by the following expression:

$$c_g\,(\mathrm{kPa}) = 0.3\ \gamma_{gab} - 4.90$$

with γ_{gab} = unit weight of the wiring mesh.

The calculated τ_{mat} has to be confronted to the shear stress applied along the checked surface, given by

$$\tau_{mat}\ (\mathrm{kPa}) = T\ /\ B_{base}$$

where T is the external shear force applied in kN.

1.4 Bearing capacity and settlements of wall foundations.

1.4.1 Shallow foundations.

1.4.1.1 *Bearing capacity.*

By the term foundation one refers to the structure fitted to transmit the load of the building and other surcharges acting on it to the underground. The global load has not to overtake the maximum shear strenght of the soil layers. If this would happen, the foundation will undergo a sudden shear failure associated to wide settlements, not tolerable by the building. The maximum theoretical load that a foundation can support immediately before the failure is termed bearing capacity.
Foundation is defined 'shallow' if the following relation is satisfied:

$$D < 4 \times B;$$

where D is the depth of embedment below the ground surface and B is the width of the foundation (B less than or equal to L, length of the foundation). Otherwise the foundation is defined a deep foundation.
The Brinch Hansen's formula has the following expression:

$$Q = c \times Nc \times sc \times dc \times ic \times bc \times gc + sq \times y1 \times D \times Nq \times dq \times iq \times bq \times gq + 0.5 \times y2 \times B \times Ny \times sy \times dy \times iy \times by \times gy \text{ (for } \varphi>0);$$

$$Q = 5.14 \times Cu \times (1 + sc + dc - ic - bc - gc) + y1 \times D \text{ (for } \varphi=0);$$

where: Nc,Nq,Ny=adimensional bearing capacity factors, given by, dwhere Nc and Nq have the same form than in the Meyerhof formula, whereas the Ny factor is given by:

$$Ny = 1.5 \times (Nq - 1) \times tg(\varphi);$$

sc,sq,sy=shape factors, given by:

in case of inclined loads:

sc = 0.2 x (1- ic) x B/L for φ=0;
sc = 1 + (Nq/Nc) x (B/L) for φ>0;
sq = 1 + (B x iq/L) x tg(φ);
sy = 1 – 0.4 x (B x iy/L);
ic,iq,iy=inclined load factors;

in case of vertical loads only:

sc = 0.2 x B/L for φ=0;
sc = 1 + (Nq/Nc) x (B/L) for φ>0;
sq = 1 + (B/L) x tg(φ);
sy = 1 – 0.4 x (B/L);

dc,dq,dy=depth factors, given by:

dc = 0.4 x k for φ=0;
where k=D/B for D/B<=1 and k=atang(D/B) for D/B>1
dc = 1 + 0.4 x k;
dq = 1 + 2 x tg(φ) x [1 - sen(φ)]^2 x k;
dy = 1.

ic,iq,iy=inclined load factorsi, given by:

ic = 0.5 - 0.5 x sqr[1 - H/(A x c)] for φ=0;
ic = iq - (1 - iq)/(Nq -1) for φ>0;
iq = [1 - 0.5 x H /(V + A x c x cotg(φ))]^5;
iy = [1 - 0.7 x H /(V + A x c x cotg(φ))]^5 for b°=0;
iy = [1 - (0.7-b°/450) x H /(V + A x c x cotg(φ))]^5 for b°>0;
where H=horizontal component of the load;
V=vertical component of the load;
b°=Tilt of the base in respect to the horizontal plane.;
A=effective foundation area ;

bc,bq,by=tilted base factors, given by:

bc = b°/147 for φ=0;
bc = 1 - b°/147 for φ>0;
bq = exp[-2 x b(rad) x tg(φ)];
by = exp[-2.7 x b(rad) x tg(φ)];

gc,gq,gy=slope factors, give by:

gc = p°/147 for φ=0;
gc = 1 - p°/147 for φ>0;
gq = gy = (1 - 0.5 x tg p°)5.

In case of structure which transmits moments to the foundation, vertical load is not centered anymore. If V is the vertical load applied to the foundation and Ml and Mb are the moments acting, respectively, along the B and the L sides, the eccentricity is given by:

eb = Mb/V;
el = Ml/V;

where eb = eccentricity along B;
el = eccentricity along L.

The assessment of the bearing capacity will be executed, using effective sizes given as follows:

B' = B - 2 x eb;
L' = L - 2 x el.

1.4.1.2 ***Settlements.***

Though the foundation load does not overtake the bearing capacity, the strains owing to the stress diffusion inside the soil mass might lead to settlement intolerable by the structure.
Settlement is due to elastic and plastic strains of the soil layers and, in case of impervious soil (silt and clay), to the slow outflowing of water from the pores (consolidation).
As the geotechnical behaviour varies from a point to another, as well as the load conditions, settlement may locally assume different values.
Settlement measured or calculated in a specific point is termed total settlement, the difference between total settlements in two or more different points is termed differential settlement.
The total settlement is given by three components:

$$Stot= Simm + Scon + Ssec;$$

where:
Simm=immediate settlement, Due to the initial strain, with no volume variation, of the loaded soil; it is usually prevalent in granular soils;
Scon=consolidation settlement, Due to the gradual outflowing of water included in the soil pores; it is prevalent in soil with low permeability (silt and clay);
Ssec=secondary settlement, Due to the viscous strain of the solid framework of the soil; it is usually neglectable compared to the other settlements.

Owing to the different behaviour of the granular and cohesive soils, the two cases have to be separately examined.

1.4.1.2.1 Settlements in granular soils.

It may be used to calculate the immediate and secondary settlements. It has the following expression:

$$Stot = C1 \times C2 \times Q \times DH \times \Sigma(Iz/E);$$

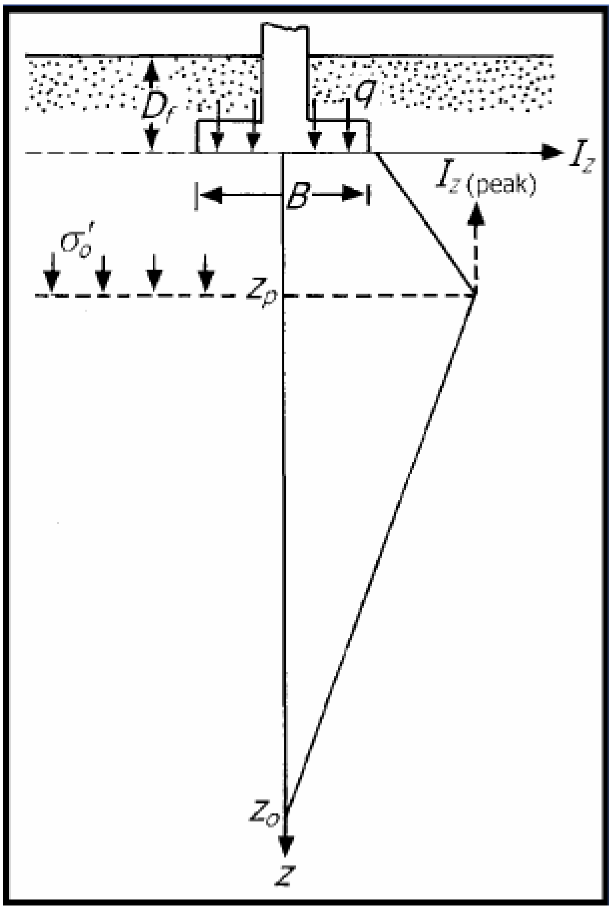
where:
Q=net load applied to the foundation;
C1=correction factor to take in account the depth of embedment of the foundation:

$$C1 = 1 - 0.5 \times (P/Q);$$

where P=effective lithostatic pressure at the depth of embedment;
C2=correction factor to take in account the secondary settlement:

$$C2 = 1 + 0.21 \times Log(T/0.1);$$

where:
T=calculation time (years);
DH=layer thickness;
E=strain modulus fo the layer;
Iz=factor to take in account distribution of the stress inside the foundation soil owing to the shallow load; it depends on the foundation shape.



	L/B=1	L/B≥10
Iz a z=0	0.1	0.2
Zp/B	0.5	1.0
Z0/B	2.0	4.0
E	2.5 qc	3.5 qc

Iz starts from a value, corresponding to the case z=0, and increases up to a peak value given by:

$$Iz_{(peak)}=0.5+0.1(Q/\sigma_0)^{0.5}$$

At the depth Z0, Iz turns null.

In case of rectangular foundation, the two former cases ($L/B \geq 10$ and $L/B=1$) are solved and one considers an interpolating value.
One suggests to use the E values proposed by the Author. This method is valid for rigid foundations only.

1.4.1.2.2 Stress diffusion beneath the foundation due to the foundation load (Newmark)

It is based on the assumption that the foundation soil can be considered a semi-infinite, homogeneous, isotropic, weightles half-space. It derives from the integration on a rectangular or square area BxL of the Boussinesq equations.
From a practical point of view, the increasing of the effective pressure, owing to the shallow load, at the depth z below the foundation, along the vertical line passing through a vertex of the area B x L, is given by:

$$pz = [Q/(4 \times \pi)] \times (m1 + m2);$$

where: m1=[2 x M x N x sqr(V) x (V + 1)] / [(V + V1) x V];
m2=atang[(2 x M x N x sqr(V))/(V1 - V)];
where M=B/z;
N=L/z;
$V=M^2 + N2 + 1$;
$V1=(M \times N)^2$

To assess the load diffusion along more verticals, the total area B x L has to be divided in smaller areas, summing then the contribution of the single sub-areas.
The Newmark method usually gives overestimated values of the stress inside the soil mass and, consequently, of the settlement too.

1.4.1.2.3 Settlements in cohesive soils

Theory of elasticity

The theory of elasticity assumes the foundation soil has a perfectly elastic behaviour. The expression is the following:

$$S = DH \times Qz / Ed;$$

where: DH=layer thickness;
Qz=Stress increase due to the the shallow load calculated at the depth corresponding to half layer.
Ed=oedometric modulus of the layer.

This method often gives an overestimated result. The calculate settlement correspond to the immediate component only, as the secondary settlement is considered neglectable. The calculated value is valid for flexible foundations only. In case of rigid foundations, the result has to be corrected, applying a factor usually sets equal to 0.93. Besides this method is applicable only when the following condition is satisfied:

$$DH < B;$$

with B=minor side of the foundation.

Edometric method

It is based on the parameters obtained by consolidation tests; it has the following expression:

$Sc = DH \times [Cc/(1 + e0)] \times Log[(Pf + dp)/Pf]$
(normally consolidated layer);

$Sc = DH \times [Cc/(1 + e0)] \times Log[(Pf + dp)/Pf]$
(overconsolidate layer with $dp<Pc$);

Sc= Dh x[Cc/(1 + e0)] xLog(Pc / Pf)+DH x [Cr/(1 + e0)] xLog[(Pf+dp)/ Pc] (overconsolidated layer with dp>Pc);

where: DH=layer thickness;
Cc=compression index;
Cr=recompression index;
Pf=effective stress at half layer;
Pc=overconsolidation stress at half layer;
dp=stress increase due to the shallow load at half layer;
e0=initial void ratio;

In a layered soil the method has to be applied to each layer and the results summed.
To calculate the secondary settlement the following expression is used:

Ss = DH x Cs x Log(1 + T);

where: Cs=secondary compression index;
T=calculation time (years).

This method is valid in case of one-dimensional consolidation. This condition is verified when:

DH<B;

with B=minor side of the foundation.
The calculated value is valid for flexible foundations only. In case of rigid foundations, the result has to be corrected, applying a factor usually sets equal to 0.93.

1.4.2 **Foundation on a pile group.**

In case of no bearing layers close to the depth of embedment of the base of the wall, one can resort to a foundation resting on a group of piles.
Two kinds of piles can be distinguished, both for the different cast-in-situ procedure and for the effect they product on the geomechanical characteristic of the soil layers: precast and cast-in-situ piles.
Apart they are considered the micropiles, which, just be cast-in-situ piles, are different from these because of some significant characteristics.

1.4.2.1 ***Vertical capacity of piles***

1.4.2.1.1 Precast plies

They are piles cast in situ without soil removing. They are usually used in from loose to medium dense granular soils, where the dynamic driving generally drives to an improving of the geotechnical characteristics. They are not recommended in cohesive soils, in which casting drives to a soil remolding with a consequently decreasing of the geotechnical characteristics. In the same way they are not usable in very dense soils or with boulders or cemented layers.
The calculation of the total pile capacity is performed summing both the contributions of the point capacity and of the skin resistance.
Three cases are distinguished.

1.4.2.1.1.1 Granular soils.

In this case the skin resistance can be assess through the expression by Burland (1973):

$$Qlat = Alat\ Pef\ K\ fw\ tg\ \delta;$$

with
Alat = lateral pile surface area;
Pef = effective lithostatic pressure given by:
Pef =Lpile x γ when Lpile < 15 Dpile;

Pef =15 x Dpile x γ when Lpile > 15 Dpile;
Lpile=pile length;
Dpile=diameter or side size of the pile;
γ =unit weight of the soil layer;
K =1-sen φ‘;
φ‘=(3/4)φ + 10;
φ =angle of shear resistance of the soil layer.
δ =angle of shear resistance pile-soil, set =20° in case of steel piles and =(2/3)φ‘ for concrete piles;
fw= correction factor in case of tapered piles, this last one measured in percentage of pile diameter (tr%) (e.g. a 5% tapering means the pile diameter decrease of 0.05 m per each meter of pile length).

setting ω(°)= arctg(tr/100)
for ω=0 (cilindric pile) fw=1;
for ω>0 (tapered pile) values of fw are giving by the following table:

φ‘	ω°	fw
φ‘<30	**ω°≤0.8**	**1+1.5 ω°**
φ‘<30	**0.8<ω°≤1.6**	**2.75 ω°**
φ‘<30	**ω°>1.6**	**2.8+ω°**
30≤φ‘<35	**ω°≤1.1**	**1+2.45 ω°**
30≤φ‘<35	**1.1<ω°≤1.6**	**2.16+1.4 ω°**
30≤φ‘<35	**ω°>1.6**	**4+0.25 ω°**
35≤φ‘<40	**ω°≤1**	**1+3.3 ω°**
35≤φ‘<40	**ω°>1**	**4.3**
φ‘≥40	**ω°>0.5**	**4**
φ‘≥40	**ω°≤0.5**	**1+6 ω°**

The point capacity is given by the following formula:

Qbase = (Abase Pef Nq)- Wpile;

with

Abase = area of the pile point;
Nq =adimensional factor by Berantezev;
$Nq = 10^m$;
$m = -0.764 + 0.076\,\varphi'$;
Wpile = pile weight.

1.4.2.1.1.2 Normally consolidated or slightly overconsolidated soils (overconsolidation ratio OCR<4).

As in the previous case, contribution of both the skin resistance and pile point are summed.
The skin resistance is given by:

$$Qlat = Ca\ Alat;$$

with Ca=pile-soil cohesion;
As to the Ca parameter, they can use the values suggested by Tomlison:

Lithology	Pinf/Dpile	Ca/C
Sabbia giacente su terreni coesivi compatti	**<20**	**1.25**
Sabbia giacente su terreni coesivi compatti	**≥20**	**0.80**
Argille molli su terreni coesivi compatti	**<20**	**0.40**
Argille molli su terreni coesivi compatti	**≥20**	**0.70**
Terreni coesivi compatti	**<20**	**0.40**
Terreni coesivi compatti	**≥20**	**0.60**

Pinf/Dpalo = interlocking ratio = ratio between the depth of interlocking of the pile inside the stiff clay layer and the pile diameter;
Ca/C=pile-soil/soil cohesion ratio

The capacity of the pile point is computed as follow:

(54)Qbase = Abase C Nc;

with
Nc = adimensional capacity factor;
One can often use a Nc value equal to 9, as proposed by Skempton.

1.4.2.1.1.3 Overconsolidated soils (OCR≥4).

It proceeds likewise the granular soil case, changing the k factor of the skin resistance expression as follow:
$k=(1-\text{sen}\ \varphi)\sqrt{OCR}$;
φ= angle of shear strength in drained condition.

1.4.2.1.2 Cast-in-situ piles

They are piles cast in situ with soil removing. They are usually used in granular soils from medium to very dense and in cohesive soil layers, where they drive to less soil remolding then the precast ones.
The computing of the pile capacity is executed as in the case of precast piles, summing contribuition of the point pile and of the skin resistance.
Appliyng the previous seen expressions, it has to take in account that, because of the remolding induced inside the granular layers by the soil removing, the angle of shear strength to be used has to be decreased as follow:

$\varphi'^{\circ}=\varphi^{\circ} - 3^{\circ}$.
with φ=angle of shear strength before remolding.

1.4.2.1.3 Micropiles.

It has to proceed likewise the cast-in-situ piles, inserting in the computing the length and the diameter of the grouting bulge instead of the length and the diameter of the pile.

1.4.2.2 ***Capacity of the pile group.***

Efficiency of s pile group is defined as the ratio between the capacity of the group and the sum of the capacities of the single piles.

$$Epilegroup = Qpilegroup / \Sigma Qpile;$$

Whereas in granular soils efficiency is usually close to 1, and in some cases even greater than 1, because of the soil densification due to the cast in site of the pile, in cohesive layers it is often lesser than 1. The main cause is the overlapping of the pressure profiles of the single piles, driving to a decreasing of their contribution to the total capacity.
A simple criterion to compute the pile group capacity in cohesive soils was proposed by Terzaghi & Peck (1948): the vertical capacity of the pile group has to be set equal to the lesser between the two following parameters:
a)the capacity given by the sum of the capacity of the single piles;
b)the capacity of the soil block with a width equal to the width of the pile group, a length equal to the length of the pile group and a depth equal to the length of the piles, given by:

$$Qgroup = Bgroup \times Lgroup \times Cbase \times Nc + 2(Bgroup + Lgroup) \times Lpile \times Clat;$$

with
Cbase=cohesion of the soil below the base of the pile group;
Clat=cohesion of the soil acting laterally to the block;
Nc=capacity factor, usually set equal to 9 (Skempton);
Lpile=pile length.

Inside granular soils, with spacing from 2.5 to 6 Dpile (Dpile=diameter or mean side of the pile), one can assume that the capacity of the pile group is simply given by the sum of the single pile capacities.

1.4.2.3 ***Spacing of a pile group.***

The pile spacing is a fundamental parameter, directly affecting the pile group efficiency. Too small or too width spacing might drastically decrease the pile group capacity. Moreover, e.g. in case of precast piles in medium or very dense granular soils, a too close spacing might drive to a reciprocal damaging of the piles.

One generally suggest a spacing widther than 3Dpile in clay, to take in account the soil remolding due to the cast in site of the pile, whereas in loose sandy soils the spacing can be reduced to 2.5 Dpile.

1.4.2.4 ***Capacity of laterally loaded piles***

1.4.2.4.1 Capacity of a single pile

The piles below the wall foundation are subject to moment and laterally loads. Therefore it has also to be assessed the lateral resistance of the foundation layers.
Here it is considered the Broms theory (1964) applied to stiff piles with bonded head, distinguishing between foundation in cohesive and granular soils.

1.4.2.4.1.1 Cohesive soils

The lateral resistance is given by:

$$(a)Rlat=9\ Cu\ Dpalo\ (Lpalo - 1.5\ Dpalo);$$

con
Cu=undrained cohesion of the soil;
Dpalo=diameter or mean side of the pile;
Lpalo=pile length.

Therefore the soil resistance has a rectangular profile, that is uniform with the depth:

$$Rz=9\ Cu\ Dpalo.$$

1.4.2.4.1.2 Granular soils

In this case (a) has to be rewritten as follow:

$$(b) Rlat=1.5\ \gamma\ Lpalo^2\ Dpalo\ Kp;$$

with
γ = unit weight of the soil;
$Kp=(1 + sen\ \varphi)/(1 - sen\ \varphi)$.

The soil resistance has a triangular profile, that is increasing with the depth:

$$Rz=3\ \gamma\ Lpalo\ Dpalo\ Kp.$$

Both to (a) and to (b) has to be applied a safety factor, as requested by the Codes:

$$Res=Rlat/Fs;$$

with
Fs=safety factor.

1.4.2.4.2 Lateral capacity of the pile group

Likewise to the case of a pile group undergoing to vertical loads, also for a pile group subject to lateral loads has to be defined the efficiency. Efficiency of s pile group laterally loaded is defined as the ratio between the lateral capacity of the group and the sum of the laterla capacities of the single piles.
Whereas in granular soils efficiency is usually close to 1, and in some cases even greater than 1, because of the soil densification due to the cast in site of the pile, in cohesive layers it is often lesser than 1.
The laterl capacity of the pile group has to be set equal to the lesser between the two following parameters:
a)the capacity given by the sum of the capacity of the single piles;
b)the capacity of the soil block with a width equal to the width of the pile group, a length equal to the length of the pile group and a depth equal to the length of the piles, given by:

Cohesive soils:

$$Rgroup=9\ Cu\ Lpile(Lgroup-Cr);$$

with
Lgroup=width of the pile group;
Cr=the lesser value between (1.5Dpile)and (0.1Lpile).

Granular soils:

$$Rgroup=1.5\ \gamma\ Lpile^2 Lgroup\ Kp.$$

1.5 Global stabilty

1.5.1 Problem definition

Procedures to analyse soil slope stability, through assesment of the limit equilibrium, consist of estimating of a safety factor relative to translational and/or rotational equilibriumof the soil volume between the ground profile and the potential slip surface imposed.
Calculation procedure takes in account the whole set of forces and moments working along a shear plane, giving an assessment of the global stability through the equilibrium equations.
A global safety factor is calculated by the ratio between the maximum available shear resistance along the collapsing surface and the mobilized strength along the same surface:

$$Fsic = Tmax / Tmob;$$

with
Fsic= safety factor;
Tmax= available shear resistance;
Tmob= mobilized strength.

At the equilibrium (Tmax=Tmob) Fsic has to be equal to 1.

1.5.2 Setting the calculation procedure

To apply the static equations to the soil slope analysis, the following conditions have to be verified:
a) verify has to be executed taking in account a section of slope having unitary width (usually 1 m), negleting lateral interactions between this slice and the surrounding ground;
b) shear resistance along the potential collapsing surface has to be expressibled by the Mohr-Coulomb law:

$$Tmax = c + \gamma\ h\ tg\ \varphi;$$

with
Tmax = maximum shear resistance of the soil;
c= cohesion of the soil;
γ= unit weight of the soil;
h= depth of the collapsing slip;
φ = angle of shearing resistance of the soil.

c)accuracy with which the geotechnical parameters are assessed, in situ or in laboratory, has to be the same: otherwise the mobilized shear resistance has to be expressed by the following way:

$$Tmob = (c/Fsicc) + (\gamma\ h\ tg\ \varphi/Fsicp);$$

with
Fsiic =safety factor related to c;
Fsicp = safety factor related to φ;

d)there has to be an homogeneous distribution of the tangent stesses mobilized (Tmob) along the potential collapsing surface, that is in every point along the hypotetical slip plane the parameters of the Mohr-Coulomb equation, c, φ, γ and h, have to have the same values.
To limit the error inserted in the calculation by this last hypotesis, the slip surface, in the most part of the procedures noted in letterature, is divided in

more slices, inside of which the homogeneous distribution of Tmob is considered to be satisfied.
By a practical point of view, slices are placed where either there is a variation of the geotechnical parameters or there is a significant changes in the topographic profile. This way to set the problem drives however to the introduction in the analytical solution of new unknown variables, related to the way in which slices interacting each other along the contact lines.
Consequently, in the calculation of the safety factor they take part the following unknown variables (n=number of slices to be considered):

a) the normal forces (N) working on slices (n unknown variables);

b) the tangential forces (T) working on slices (n unknown variables);
c) the points of application of the normal and tangential forces on slices (n unknown variables);
d) the horizontal forces acting on the dividing surfaces between contiguous slices (n-1 unknown variables);
e) the vertical forces acting on the dividing surfaces between contiguous slices (n-1 unknown variables);
f) the points of application of the forces d) and e) (n-1 unknown variables);
g) teh safety factor Fsic (1 unknown variable).

Then solution involves an overall set of 6n-2 unknown variables. Actually they are available:

a) 3n equilibrium equations;
b) n equations of this sort:

$$T = (c\ l + N\ tg\ \varphi)/Fsic;$$

with
l = length of the slice;

which link, for each slice, the unknown variables N,T and Fsic.

c) n equations given imposing the points of application of N, T on the central point of the slice base.

In all, 5n equations are available to analytically solve the problem. To get Fsic, there obviously would need as many equation as the unknown variables.
So that the problem be statically determined, and consequently solvable, they actually miss still n-2 equations. These equations can be gotten inserting in the analysis some simplifying hypothesis, generally regarding the distribution of forces along the dividing surfaces of contiguous slices.
The several available solving procedures differ each other essentially for the chosen simplifying hypothesis about the force distribution.

1.5.3 Solving by limit equilibrium methods

1.5.3.1 Bishop (simplified)

By the Bishop method the condition that the vertical forces working along the dividing surfaces of contiguous slices are negligible is imposed. Consequently slices interact among them through horizontal forces only.
It is a method based on the equilibrium of the moments.
One supposes the potential slip surface be circular.
The maximum shearing resistance available along the potential sliding surface is given, for each slice, by

$$Ti\ max = Xi / (1 + Yi / SF);$$

with $Xi = (c + (g \times h - gw \times hw) \times tg\ \varphi) \times dx / \cos \alpha$
where gw = water unit weight;
hw = height of the water table with respect to the slice bottom;
dx = horizontal length of the slice;
α = slice inclination with respect to horizontal plane.

$Yi = tg\ \alpha \times tg\ \varphi$

The mobilized shearing strenght along the shearing plane is, for each slice, given by:

$$Ti\ mob = Zi$$

with $Zi = g \times h \times dx \times sen\ \alpha$

The slope safety factor is expressed as follow:

$$SF = \sum(i=1-n)\ Ti\ max / \sum(i=1-n) Ti\ mob$$

One can note that the safety factor SF, the variable to be determined, appears in the numerator too through the Tmax expression. Consequently one cannot directly determine SF.
Procedure to be adopted should be iterative, till obtaining convergence over a practically constant value of SF.
Steps are the following:

1.an initial value of SF is imposed (for example given by Fellenius method) and a first SF value is calculated;

2.the resulting new value of SF (SF') is compared to the initial value;

3.if the difference exceeds a prefixed value (e.g. SF'-SF> 0.001), one come back to step a), inserting, instead of the first value SF, the new calculated value;

4.if the difference lays inside the prexifed limit, calculation is aborted and SF' is the searched value.

Procedure generally needs from 4 to 8 iterations to converge.
The Bishop method requires the following two conditions be respected:

- s' = (g x h - gw x hw - c x tg α / SF)/(1+Y / SF) > 0

  with s' = normal stress on the slice bottom;

- cos α x (1 + Y/SF) > 0.2.

If not, method may drive to unrealistic SF.
Method has preferably to be applied in case of homogeneous soil slope, both by lithological and geotechnical point of view.Use of this method in not recommended in case of highly overconsolidated layers.
Comparing simplified Bishop to its complete version, one can get a maximum difference in the SF values not exceeding 1%. With respect to other more rigorous methods, as G.L.E., deviation do not exceed 5%, except in case of SF<1, of scarce practical importance.