REFERENCE MANUAL

MACCAFERRI

1. USER'S MANUAL (Please see User's manual)

2. REFERENCE MANUAL

2.1 INTRODUCTION

The Macstars W, has been developed to check the stability of gabion and reinforced soil support works, that is structures which provide slope stability using reinforcing units that are able to absorb the tensile stress. Furthermore, this program allows the user to conduct the stability checks using the Limit Equilibrium Method even considering unreinforced slopes and without support works.
The new version of the MACSTARS W design program also allows calculation of the critical sliding acceleration, i.e. the pseudo-static acceleration value which renders unstable natural slopes and gabion and reinforced soil support works, which may be compared to a rigid block.
The type of check to carry out in relation to the soil potential failure mechanisms, the behaviour of the reinforcing units, the type of loads to be considered, are fundamental aspects which will be shortly and separately illustrated herein after.
Before illustrating the Macstars2000 calculation method, hereunder we provide some fundamental definitions used in this program.

2.2 BASIC DEFINITIONS (see fig.1)

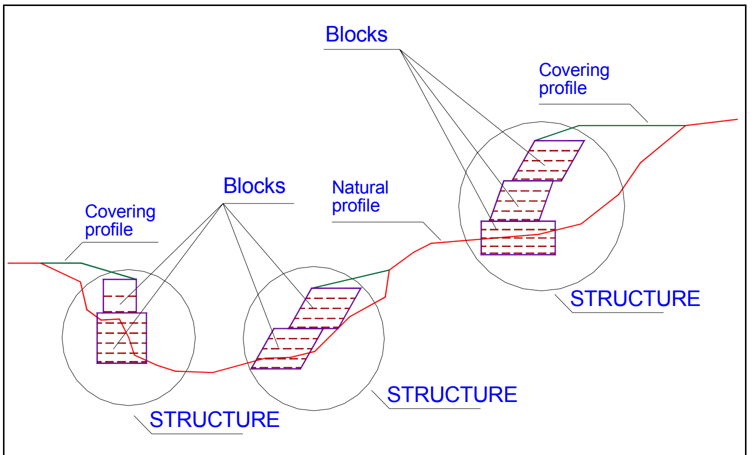


Figure 1

Original slope: original soil profile, before the installation of the designed reinforcing units.
Retaining structure: sequence of reinforcing structures called blocks; a slope may consist of one or more reinforcing structures; the reinforced structure may be superficially covered with filling soil.
Support works (gabions) : continuous sequence of gabion structures, called walls; a slope may, therefore, comprise several support works; the works may be covered by top soil
Covering filling: the profile of the soil laid over the retaining structure to join one reinforced block to the one right above or to join the retaining structure and the natural slope
Block: single reinforcing structure consisting of the structural embankment, the reinforcing units, and the backfill. The block facing may also obtained by special units, Terramesh System, that have gabions on the front face
Layer: structure formed by gabions placed on the same plane (horizontal or sloping)
Structural embankment: the soil used for the reinforcing block, distributed in layers between the reinforcing units, mechanically compacted to improve its mechanical and resistance characteristics or between the wall and original slope (if present).
Backfill: soil layer used to fill the space between the reinforcing block and the original slope
Reinforcement unit: reinforcing element resistant against tensile stress due to the friction which develops with the soil, installed in horizontal layers; it can be either the main reinforcing element and in this case it is provided with a fold along the downstream face or the secondary element which is installed between the folding face of the upper and lower main element; the secondary unit is always longer than the main one.

Facing (Wall Batter): the free face of the block or wall facing downhill
Gabions: 1-structure filled with stones, which forms the front face of the reinforcement works used for drainage and erosion control purposes or to give to the front face a higher stiffness in case of a vertical retaining walls
2-retaining wall.
Wrapped length: part of reinforcement unit extending from the upper front face into the backfill for a length of 50-100 cm
Anchorage length: the reinforcements length behind the failure surface
Pullout: the reinforcing unit maximum pullout resistance along the anchored segment or within the unstable soil portion

2.3 TYPES OF STABILITY ANALYSIS (CHECKS)

Macstars2000 allows the user to conduct the following calculation:

- Global stability Analysis
- Internal stability Analysis
- internal sliding analysis for gabion structures (sliding analysis between a layer of gabions and the layer above)
- Wall Stability Checks
- Sliding stability Analysis
- Settlements calculation

2.3.1 Overall stability check

The overall stability check, global stability or basic stability, is the stability analysis of a reinforced or un-reinforced slope carried out by using the limit equilibrium method. It can be conducted to check the stability of a non-reinforced slope without a gabion wall, prior to considering the reinforcements. For design purposes this stability analysis is required to evaluate the retaining work stability against potential deep-seated sliding mechanisms as well as sliding mechanism external to the gabion wall and to the reinforcing units (figure 2).

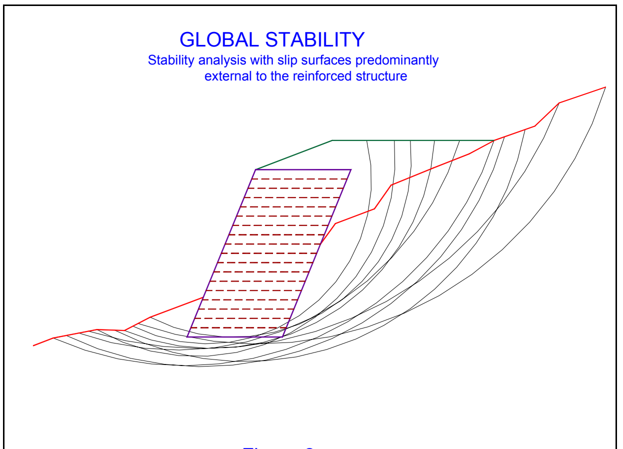


Figure 2

2.3.2 Internal stability check

The internal stability check (or slope stability) allows the user to determine the design of the retaining structure, that is the reinforcing units required (type, spacing between reinforcing unit, length, etc…). According to this type of stability analysis the surfaces of potential sliding originate from the toe of the reinforcing structure and, passing through the reinforcement, terminates uphill (figure 3).

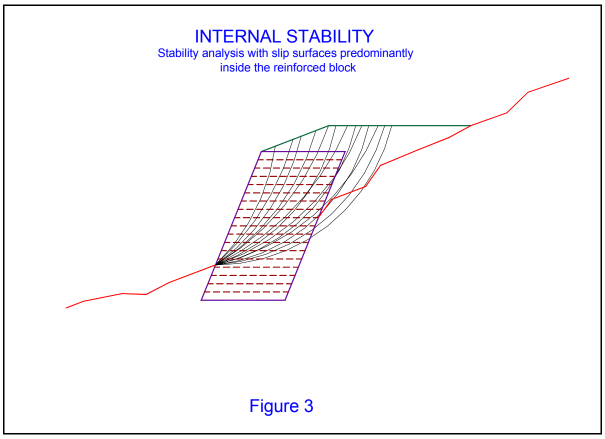


Figure 3

2.3.3. Stability check of the structure as a retaining wall (Sliding, Overturning and Foundation load-bearing capacity)

In conducting this type of stability analysis the entire retaining structure, or part of it, is considered as a monolithic wall consisting of blocks, which form the retaining structure, itself. During the stability analysis the wall may be considered as being formed by all structural blocks (considered as structural embankments) forming the retaining structure or by all blocks above the specified block.
In order to consider the sequence of selected blocks as a monolithic wall, a geometrical condition of mean slope (inclination) of the reinforcing block must be satisfied (figure 4): **it must be higher than or equal to 70°**. The program determines the mean slope considering the straight line connecting the right lower corner of the first block (figure 4, point A) with the upper right corner of the last block of the structure to check (figure 4, point B)

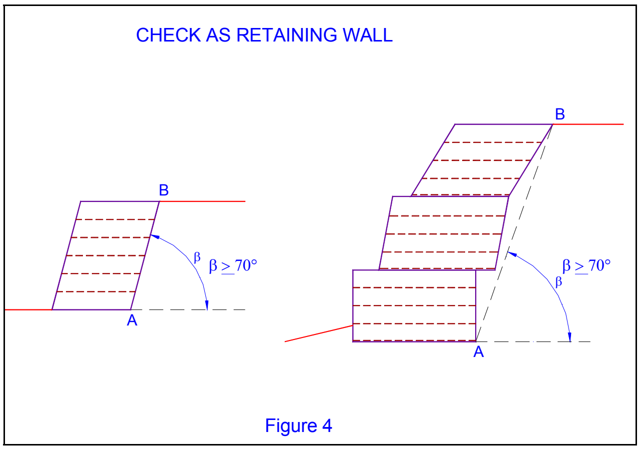


Figure 4

In this analysis, the gabion support works are considered to be a monolithic structure.
The stability check of the structure (whether it is reinforced soil or a gabion wall) as a retaining wall consists of the three classical stability analyses conducted on retaining walls (figure 5): check against overturning (A), check against sliding (B), check against the foundation bearing capacity (C). For this last stability check, the value of the ultimate soil pressure at the base of the wall can be provided by the user or can be automatically calculated by the program as described in detail herein after.

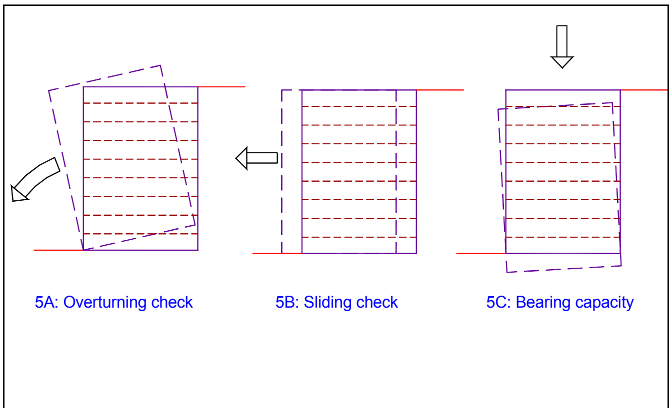


Figure 5

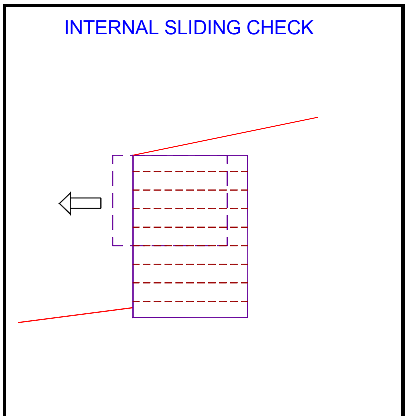


Figure 6

2.3.4 Check against sliding

This type of stability analysis is conducted to check the stability (of the entire retaining work or part of it) against sliding along an horizontal plane selected by the user (figure 6), using parameters (cohesion, friction angle on the sliding surface) selected by the user depending on the type of contact at the base.

2.3.5 Internal sliding analysis (analysis of sliding between a layer of gabions and the layer above)

This analysis checks the stability of the works (either as a whole or of a part) to sliding along a contact plane beneath a layer of gabions selected by the user, with analysis parameters (cohesion and angle of friction on the sliding surface) which, only for the base layer (the so-called first layer), may be selected by the user depending on the type of contact with the base layer.

2.3.6 Check against soil settlements

Macstars2000 allows the user to calculate the settlements induced by the installation of a reinforced soil structure. The construction soils (structural embankment, backfill, upper soil covering) are considered as loads, which induce a change in the stress distribution.
Therefore, different models of elasticity (depending on the type of soil) are used to calculate the soil failure induced by the applied loads (see par. 2.10).

2.3.7 Stability Analysis checks for a given surface.

This analysis is conducted by the user, who inputs the coordinates of the potential failure surface.

2.4 BEHAVIOUR OF REINFORCING UNITS (reinforced soils)

The reinforcing units are structural elements, which behave as follows:
1) The reinforcing units are resistant against tensile stress
2) The tensile stress within the reinforcing units can develop due to the adherence between the reinforcing unit and other materials (soil or other reinforcing units) located immediately above and below it.
3) The reinforcing units provide a stabilising force in the area where they intercept a sliding surface, that is the area of the slope in which develops the shear stress, which induces a deformation or extension of the reinforcing unit.
4) As the deformation increases, the strength provided by the reinforcing unit increases as well until it reaches a maximum value which, in relation to the geometry of the considered problem, can be: the reinforcement unit tensile resistance, the pull-out resistance along the anchorage area or the pull-out resistance within the unstable soil portion.

In order to consider both common simplified methods and more complex realistic reinforcement behaviour, two analytical models are provided by the software:
- rigid model
- deformative model (Displacement Method)

2.4.1 Rigid model

The rigid model procedure assumes that any reinforcement crossing the analysed potential sliding surface provides a resisting tensile force (reduced by partial safety coefficients), independently from the stiffness values of the reinforcing elements. For each reinforcement the following conditions must be checked:

- a minimum anchorage length (normally this length is 15-20 cm)
- the pull-out resistance in the anchored zone
- the pull-out resistance in the unstable soil zone

In the first case, an anchorage length smaller than the minimum established reduces the reinforcing unit tensile strength.

In the second and third case, the tensile stress in the reinforcement is limited to the lower of the two pullout values.

The calculation of the pull-out force is conducted according to the following procedure, which is based on the consideration that at all points of the reinforcement the ultimate condition ($?_u$) is reached

External pull-out (anchorage area)

The anchorage area is divided into segments and for each segment the ultimate tangential stress (τ_u) must be calculated according to the following equation:

$$\tau_u = f \cdot \sigma_v$$

Where: f = the total friction coefficient of the reinforcing element on the upper and lower materials in the segment considered, which can be reinforcement over reinforcement (f_{rr}) or reinforcement over soil (f_{tr})

σ_v = vertical stress acting on the segment considered, obtained by the following equation:

$$\sigma_v = (W + P_v - U) / dx$$

W = total weight of the upper soil column

P_v = vertical component of the uniformly distributed surcharge load acting uphill

U= pore water pressure

dx = width of the segment considered

The integral of the ultimate tangential stress provides the ultimate pullout resistance of the reinforcement. A safety coefficient defined by the user can be added to this value.

Internal pull-out

In case of secondary reinforcing elements the calculation procedure of the ultimate pullout force is the same as the procedure used to calculate the external pullout.

The length of the reinforcing element within the unstable soil block is divided into segments and for each segment the value of the ultimate tangential stress (τ_u) is calculated by using the following equation:

$$\tau_u = f \cdot \sigma_v$$

Where the meaning of the symbols used is the same as in the case previously illustrated. The integration of the ultimate tangential stress provides the value of the ultimate internal pullout stress.

In case of main reinforcing elements the resistance contribution due to the wrapped length of the reinforcing unit must be added. This contribution (F_0) can be calculated by the sum of two contributions:

$$F_0 = F_1 + \Delta F$$

Where F1 is the contribution which generates the pullout stress on the tail (horizontal) whereas ΔF is the additional contribution which takes into consideration the stress acting on the sub-vertical portion, adjacent to the wall batter.

F_1 is calculated by using a procedure similar to the one used for the external pullout (integration of the ultimate tangential stress), whereas ΔF is calculated assuming that the analysed area has a semi-circular configuration according to the following equation:

$$\Delta F = F_1 \cdot \pi \cdot f_{tr}$$

A safety coefficient, defined by the user, can be added to the value of the total ultimate pullout resistance.

2.4.2. Deformative model (Displacement Method)

According to this model the resistance produced by a reinforcing unit crossing the potential sliding surface is calculated by taking into consideration:

- the stress-strain behaviour of the reinforcing element assumed as an isolated unit
- the stress-strain behaviour of the contact area between the reinforcing element and the upper and lower materials

Given a displacement δ (horizontal component of the slope overall displacement), the calculation of the stress acting on the reinforcing unit is based upon the following assumptions:

1) SOIL-REINFORCEMENT INTERACTION
The relation between shear stress (τ) and displacement (δ) is hyperbolic, as for the behaviour of piles with lateral friction, and it is therefore defined by the equation (figure 7)

$$\tau = \tau_u \cdot \frac{\delta}{\delta_e + \delta}$$

Where:
τ = shear stress mobilised for a displacement δ (kPa)
$\tau_u = f\sigma_v$ = ultimate shear stress (kPa)
$\delta_e = k\,\sigma_v$ = elastic displacement, obtained with the initial tangent (m)
k= elastic sliding parameter obtained from direct shear tests as δ_e/σ_v (m^3/kN)
σ_v =vertical pressure during tests (kPa)
f= friction coefficient soil-reinforcement: Macstars2000 considers two surfaces (up and down)

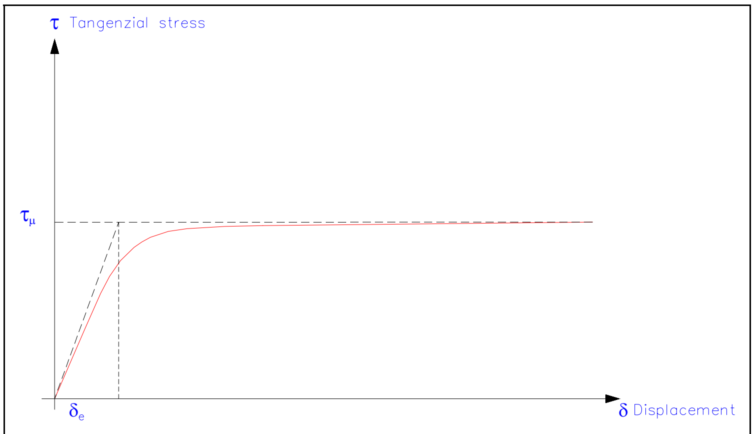


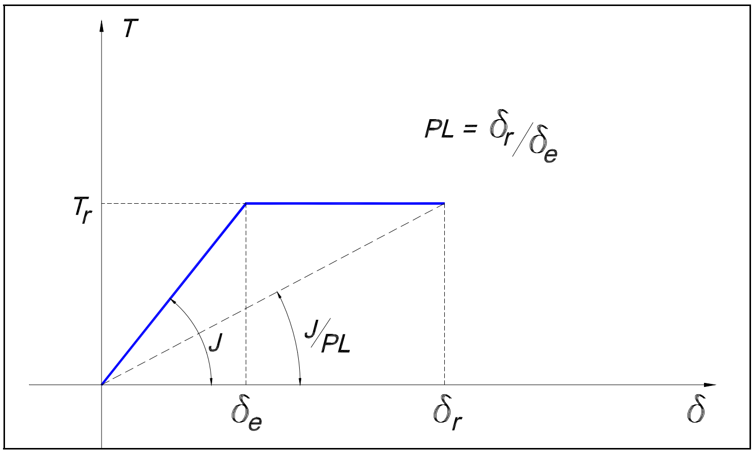
Figure 7

2) STRESS-STRAIN BEHAVIOUR OF THE REINFORCEMENT
The stress-strain behaviour of the reinforcing element is given by the classical equation of elasticity:

$$dL = \frac{N \cdot L}{E \cdot A}$$

Where:
N= average axial stress on the reinforcement (kN)
L= length of the reinforcement stretch (m)
E= modulus of elasticity (kN/m^2)
A= reinforcement section per linear meter (m^2/m)
dL= elongation of the stretch of the reinforcement (m)

As per the stress-strain characteristics of each reinforcement, reference is made to the parameters obtained from pullout tests, where the reinforcement’s deformation δ occurring at different levels of the pullout force T are measured for various soil types and σ_v.



T_r = conventional unit breakage force (kN/m)
J=EA= linear stiffness of the reinforcement (kN/m), which is constant in the elastic stretch, thus δ=T/J
PL= δ_r / δ_e= plastic sliding parameter, given by δ_r / δ_e (adim.)
δ_r = displacement at breakage (mm)
δ_e = elastic displacement (mm)

The parameter of the model which are obtained from direct shear and pullout tests and used by MACSTARS W are K, f, T_r, PL and J.
The procedure, which leads to the calculation of the stress acting on the reinforcing unit, is iterative according to the following steps:

1) The external length of the reinforcement is divided in sections
2) A value of N_i at the edge of the reinforcing unit is assumed
3) The program calculates the tangential stress (τ_1) acting on the first segment of the reinforcement element according to the displacement considered
4) Using the above stress value, the program calculates dN_1 (which is the variation of the axial stress on the first segment of the reinforcing unit) equal to the tangential stress τ_1 by the length of the segment of the reinforcing unit
5) The mean stress N_{med} = N - dN_1/2 is calculated
6) Then the elongation of the segment produced by the effect of N_{med} is calculated
7) Then the mean displacement of the reinforcing unit is calculated (δ_{m1})
8) Using this value, the program will repeat the calculation starting from point 3 up to point 7
9) This calculation procedure will provide the value of the tangential stress acting on the slice 1 (τ_1), the displacement in the terminal point of the segment (δ_2), the contribution to the axial stress acting on the first slice (dN_1), the initial axial stress acting on the next slice N_2 = N - dN_1
10) Then the next segment is checked by repeating steps no. 3-4-5-6-7-8-9
11) This calculation procedure is conducted for all segments and it is interrupted when the value of the displacement is null or negative (in this case the segment is divided into parts)
12) At this point all contributions dN_1 acting on each slice are added up and the value obtained is compared to the initial value N_i. If the difference is less than 5 % of the tensile strength, the calculation ends and the value N is assumed to be as the mean between the sum of all dN_1 values and the initial value N_i, then the initial value N_i is modified and the calculation is repeated starting from point 2.

If the final value N is bigger than the tensile strength and the reinforcing unit is considered as having a plastic section, then the calculation is repeated considering a fixed value N equal to the value of the tensile strength in order to check if a point of null displacement is obtained as per the previous procedure using a value of J (stiffness) reduced by a coefficient fixed in the reinforcement data-base. If this point is not reached the reinforcing unit is considered under breaking conditions.
The reinforcement reaction value obtained is eventually compared to the internal pull out force.

2.5 STABILITY ANALYSIS CONDUCTED USING THE LIMIT EQUILIBRIUM METHOD

The overall and internal stability checks refer to the limit equilibrium method.

The soil block subject to failure is divided into slices and for each one the program calculates the acting forces: external forces, weight, forces acting at the base of each slice and shear forces acting along the slices interface.
The number of unknown is bigger than the number of equilibrium equations available and therefore the problem is hyperstatic. In order to obtain a solution, it is necessary to simplify it. This problem has been analysed by different authors, who, by adopting different assumptions, have obtained different solutions: Fellenius, Bishop, Janbu, Spencer, Morgensten and Price, Sarma and others.
All the developed methods use common assumptions:

- the slope is analysed in conditions of plane deformation that is, the longitudinal dimensions are assumed to be greater than the transversal ones in order to neglect the border effect.
- the safety coefficient acting along a surface is assumed as the factor by which the parameters of shear resistance must be divided in order to bring the slope to a condition of limit equilibrium and it is assumed to be constant along the entire potential sliding surface
- The equilibrium of the entire soil block is analysed as the sum of the equilibrium conditions of each single slice.

Some characteristics of MACSTARS W code are described hereunder.

2.5.1 Methods used by MACSTARS W

Macstars calculation code employs Bishop and Janbu simplified methods.
Both methods refer to the Mohr – Coulomb failure criterion:

$$\tau = c + (\sigma - u)\ \tan(\phi')$$

Where: τ = maximum tangential stress
c = cohesion
σ = total normal pressure
u = pore water pressure
ϕ' = friction angle

Characteristics of the simplified Bishop method

- It can be applied only for circular or almost circular surfaces, that is those surfaces that are considered as failure circular surfaces adopting a fictitious centre of rotation.
- The forces interacting between the slices have only a horizontal direction
- The safety coefficient is calculated by the equilibrium against rotation around the centre of the circumference
- It does not satisfy the global equilibrium in the horizontal direction

Characteristics of the simplified Janbu method

- It can be applied to any surface type
- The forces interacting between the slices have only a horizontal direction
- The safety coefficient is calculated by the equilibrium against vertical and eventually horizontal translation
- It allows to take into consideration the vertical shear forces of interaction between the slices by applying to the previous safety coefficient a correction factor which depends on the problem's geometry and the type of soil
- It does not satisfy the global equilibrium of the soil wedge against rotation

Depending on the behaviour of the reinforcing units a stability check can be conducted by the rigid method or by the displacement method
The displacement method is further divided into the method with assigned displacements or the method of incremental displacements

Rigid Method

It is based upon the assumption that the reinforcing units behave as rigid structures

Displacement method

It is based upon the assumption that the reinforcing units behave as structures subject to deformation depending on their linear stiffness.

This method can be applied in case of a rotational shape of the sliding surface. Therefore it can be used with both Bishop and Janbu methods (at least for a given almost-circular sliding surface).
During the calculation procedure the program utilises a deformation value, which when multiplied by the length of the sliding surface, provides the value of the displacement to adopt.
This displacement must be considered as the modulus of the displacement vector, constant in each point of the sliding surface and tangent to the sliding surface itself.
Then the program calculates the horizontal component of this displacement, which is the force acting on the reinforcing element in the deformative model.

Displacement Method: Assigned Displacement option
According to this method the user provides the value of a deformation with which, during the calculation, the displacement for each sliding surface as previously detailed can be obtained.

Displacement Method: Incremental Displacements option
By this method the user defines a maximum deformation, and therefore a maximum displacement, for each surface.
Within the displacement range which goes from 0 up to the maximum displacement, the program searches and analyses different situations:

1. null displacement
2. displacement at Fs=1.0
3. displacement at Fs=Fsmin (minimum safety coefficient provided by the user)
4. displacement at Fs=Fsmax (maximum safety coefficient)
5. displacement equal to the maximum value assigned by the user (by the maximum deformation)

Item n.1 refers to the situation in absence of reinforcement (or reinforcement with a null displacement).
Item n.2 refers to the situation where the slope deformation is allowed to reach the equilibrium condition (Fs=1); if the slope has (in absence of reinforcing units) FS>1.0, then the program will not carry out the calculation since it is not necessary to mobilise the reinforcing units.
Item n.3 refers to the situation where the slope deformation allows to obtain a minimum safety coefficient (Fsmin) assigned by the user. Even in this case if the slope has (in absence of reinforcing units) Fs>Fsmin, then the program will not carry out the calculation since it is not necessary to mobilise the reinforcing units.
Item n. 4 refers to the situation where the slope deformation allows the user to obtain the maximum allowable safety coefficient within the range of the slope deformations assigned by the user. Since the function relevant to the safety coefficient may provide different relative maximum values, the iterative procedure, which leads to Fsmax, can provide just a relative maximum value and not an absolute value.
Item n.5 refers to the situation of the slope's maximum deformation assigned by the user.

For the situations illustrated in items n. 2-3-4-5- and for each analysed surface the vector of the stresses acting on the reinforcing units is memorised and for each situation and for each reinforcement the calculation determines the maximum stress and the relevant associated deformation.

2.5.2 Generation of the failure surfaces

The user can run Macstars2000 to determine a sliding surface by providing the surface co-ordinates (this procedure can be adopted when information is available on the position of the sliding surface) or to search randomly for the potential sliding surface, that is a surface which has the minimum safety factor and is the most probable surface which can induce the slope failure.
The generated surfaces can be:

- circular surfaces
- random polygonal surfaces

The calculation method adopted is: Bishop method for circular surfaces and Janbu method for circular or random surfaces

In case of a given surface, either Bishop or Janbu methods are possible but in this case the surface must proximate a circumference arch, otherwise the analysis will not be correct.
The user drives the search for the critical surface by providing some geometrical parameters such as:

- the extension of the segment from where the surfaces originate
- the extension of the segment where the surfaces end
- the magnitude of the angle from where the surfaces originate
- the length of each segment of the sliding surface
- a minimum elevation below which the surfaces cannot extend
- a geometrical profile within which the surfaces cannot enter (for example a bedrock profile)

The final result can depend upon such choices; therefore it is advisable to conduct the calculation several times using different parameters.
The user can also select how many surfaces to generate.
Each single surface is generated by considering successive segments (whose length is provided by the user) whose inclination is determined at random, but partially controlled due to the imposed values.

2.5.3 Sub-division into slices

Once a potential sliding surface has been determined, the soil block subject to failure is sub-divided into slices, according to the following parameters:

- discontinuity of the sliding surface
- discontinuity of the stratigraphic and geometrical profiles
- discontinuity of the levels of the water table
- position of the loads

2.5.4 Surcharge loads

The surcharge loads acting on the slope are:

- uniform surcharge loads
- linear loads
- point loads repeated at regular intervals
- isolated point loads
- loads generated by the presence of tiebacks
- dynamic loads due to seismic actions

Other forces are implicitly considered such as water table (see the section water table):

a) pressures acting on the soil block. When the water table is external to the soil profile.
b) filtration forces acting on the slope in case of inclined water table.

Uniform surcharge loads

The program allows the user to consider uniformly distributed surcharge loads by providing its external surface, the value and inclination of the surcharge load acting on the soil block. Such loads acting on the top of the slices are then transferred to their base without any lateral diffusion (figure 8). In case of inclined surcharge loads, the program divides them into two components, vertical and horizontal.

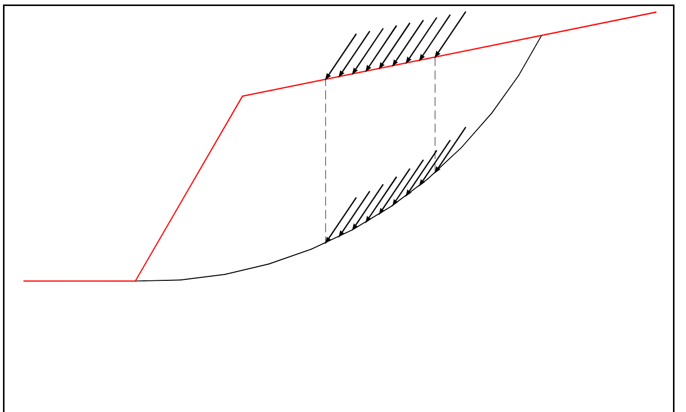


Figure 8

Linear loads acting on the longitudinal direction with respect to the slope

It is a linear load acting on the longitudinal direction with respect to the slope, which appears to be a point surcharge load in the transversal section. Such a load spreads down in depth (figure 9) according to an angle of about 27° (angle given by the ratio 1:2) from the direction of the load (therefore 54° overall)

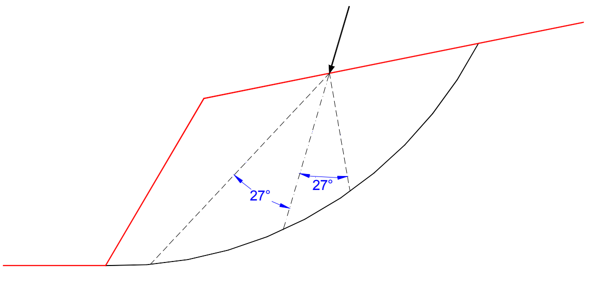


Figure 9

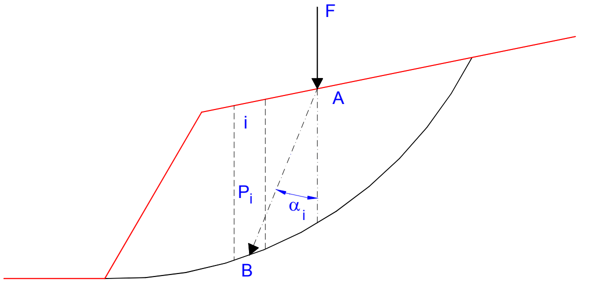


Figure 10

The value of the pressure on each slice resulting from the application of the load (F) is defined by the following procedure (figure 10):

- given a slice i, the program calculates the angle (α_i) formed by the direction of the load and the line connecting the point of application of the load (A) with the centre of the base slice (B); if such an angle is lower then the one indicated above the procedure will continue;
- the program calculates the distance (d_i) between points (A) and (B);
- the radial pressure (σ_i), that is the pressure acting on the direction AB, at the base of the slice is calculated with the Flamant equation (Morlier and Tenier 1982):

$$\sigma_i = \frac{2\ F\ \cos(\alpha_i)}{\pi\ d_i}$$

- The resulting radial force (P_i) at the base of the slice is therefore obtained multiplying the pressure by the length of the slice.

The forces thus calculated are modified in their intensity to guarantee the equilibrium of the system of forces formed by the applied force F and by the radial forces P_i, according to the following procedure:

- The program calculates the components of all radial forces Pi in the parallel (P1) and perpendicular (P2) directions of the load;
- the forces are distinguished in P1a and P1b, P2a and P2b where the index b is referred to forces for which the component P2 is negative;
- the program operates a system which allow equilibrium of the acting forces:

$$a\,[P1a] + b\,[P1b] = F$$
$$a\,[P2a] + b\,[P2b] = 0$$

- the application of the coefficient a and b respectively to the forces produces a set of radial forces P_i in equilibrium with the applied force

Point loads repeated at regular intervals
These are point loads, which are repeated at regular intervals in the third dimension (longitudinal direction).
Such a type of load is calculated according to a procedure, which at first considers the transformation of the surcharge point loads into a linear load (longitudinal distribution) and then the application of the procedure described in the previous item.
The longitudinal distribution takes place according to the following scheme:
- the program determines (figure 11) the distance (Zs) between the point of application of the load (A) and the failure surface (B)
- then the program determines (figure 12) the distance (Zcr) where the cones of longitudinal distribution of the loads with an angle equal to about 37° (angle given by the ratio 3:4) intersect (Zcr=2/3*l, being l = the load inter-axis)
- then the program determines (figure 13) the width of the slip surface intersected by the longitudinal diffusion of a single load (L)
- for Zs smaller than Zcr the equivalent linear load is given by Q/L
- for Zs greater than Zcr the equivalent linear load is given by Q/l

Once the point loads repeated by a constant inter-axis have been transformed into a linear load (longitudinal distribution), the transversal distribution is determined as described in the previous item.

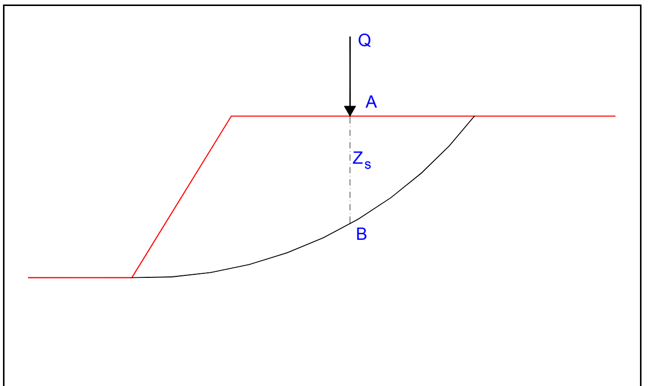


Figure 11

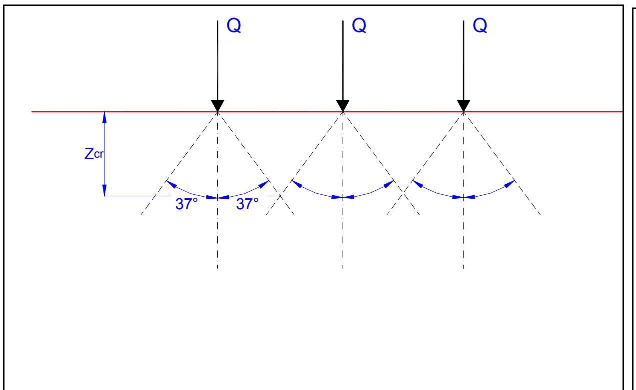


Figure 12

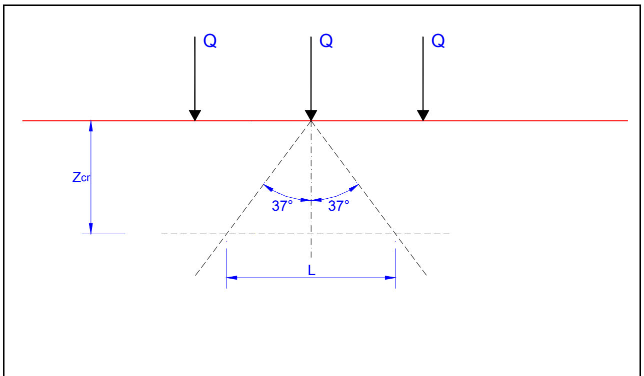


Figure 13

Isolated point loads
These are isolated loads, which do not repeat in the third dimension. Such a type of load is transformed into a linear load by a longitudinal distribution:

- The program determines (figure 11) the distance (Zc) where the load crosses the failure surface
- Then the program determines (figure 13) the width of the failure surface affected by the longitudinal distribution of the load with an angle equal to about 37° (angle given by the ratio 3:4) in correspondence of the failure surface (L)
- The point load Q is transformed into an equivalent linear load (Q/L)

Once the point load has been transformed into a linear load (longitudinal distribution), the transversal distribution is determined as described in the previous item.

Tiebacks

The load generated by the presence of the tiebacks is considered as a linear force according to the distribution of the load on the inter-axis of the reinforcing units.
At the beginning of the calculation procedure the program (figure 14) checks that the total length of the reinforcing units is such that they cross the sliding surface (the user must make sure that the tieback anchorage is external to the sliding surface)

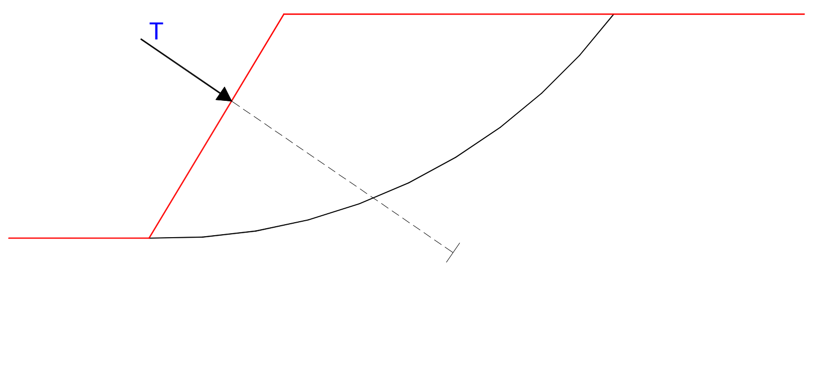


Figure 14

In this case the program will operate a distribution of the load with an angle of 90° with respect to the direction of the reinforcing units (for a total of 180°) using the same procedure adopted for the linear loads.

Dynamic loads due to seismic actions

MACSTARS W refers to the pseudo-static method to conduct the calculation procedure of seismic forces, introducing into the calculation mass forces with a horizontal and vertical direction, obtained by multiplying the total weight of each single slice by the two coefficient of seismic intensity, (horizontal and vertical) expressed as percentage of g.
Positive values of the seismic intensity coefficients generate forces oriented in an outward direction from the slope and towards the top.

2.5.5. Soil reinforcement

The presence of a reinforcement is assumed in the calculation procedure by considering a stabilising horizontal force directed into the slope (figure 15) at the point of intersection between the reinforcement and the sliding surface.

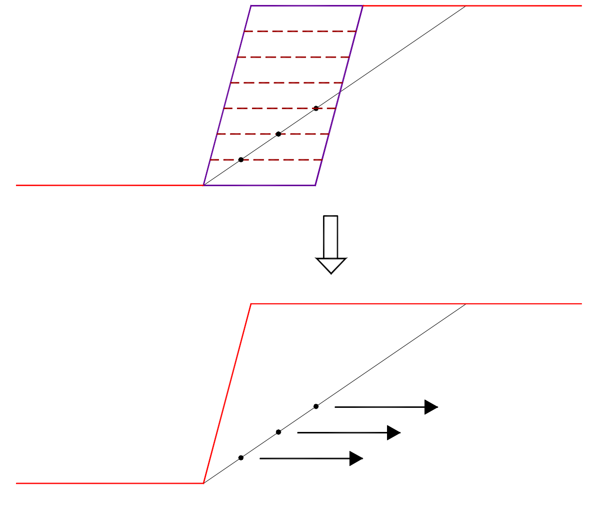


Figure 15

As for the calculation of this force, refer to paragraph 2.4 in which the different calculation procedures adopted are fully described.

2.5.6 Water Tables

The presence of one or more water tables in the soil is determined by considering lines defined by the following points:

- The point's abscissa
- The point's ordinate
- Lower ordinate of validity of the underground water table
- Pressure acting on the underground water level

The first two data refer to the underground water free surface.
The user can consider water surfaces under pressure and in this case for each point of the free surface, the acting pressure must be provided as well.
Perched water tables can also be considered.
The calculation involves the determination of other values related to the water table presence:

- weight of the slice
- pore water pressure at the base of the slice
- forces acting on the soil free surface
- forces related to the inclination of the water table (filtration forces)
- forces related to a water table which impacts upon the structure (hydrostatic thrust)

The user must take into consideration that the calculation of the interstitial pressure can also consider the parameter Ru (pore pressure parameter) as described below.

Calculation of the slice's weight

In calculating the weight of the slice, the presence of a phreatic surface within the slice involves the use of the natural unit weight of the slice portion outside the phreatic surface and the saturated unit weight of the submerged portion of the slice.
The calculation of the values necessary to determine the volumes is conducted along the mean section of the slice; the obtained values are applied to the entire slice.

Calculation of the pore water pressure at the base of the slice

The pore water pressure at the base of the slice (u) is calculated in order to determine the pressure acting at the base of the slice (2.5.1).
The calculation of the pore water pressure is rather simple for a horizontal phreatic surface, since the program can apply the equation of hydrostatic pressure (figure 16, point A_1):

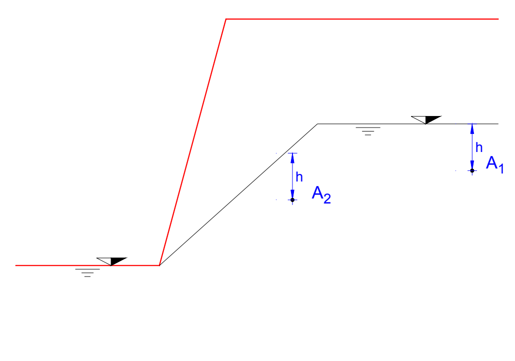


Figure 16

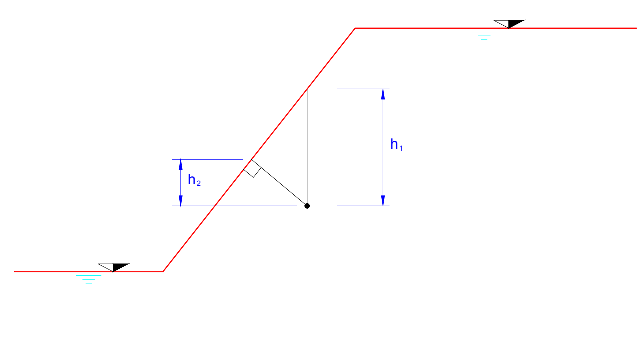


Figure 17

$$u = \gamma_w \cdot h$$

where: γ_w is the weight of the water column

h is the difference in depth between the free surface and the base of the slice, possibly increased by the pressure acting on the free surface

In case of an inclined water table the use of the hydrostatic pressure is definitely conservative and can be excessively penalising (figure 16, point A_2). It should be necessary to consider a flow network and calculate the pressure with respect to the depth of the correspondent equi-potential at the base of the slice, but unfortunately this calculation is rather complex for the purposes of this program.
Therefore the following method has been adopted (figure 17):

- given a slice, the height h_1, corresponding to the hydrostatic height (along the vertical direction) is considered
- the program then determines the depth of point B, this being the toe of the perpendicular in point A (centre of the slice base) to the phreatic surface profile
- then the height h_2 equal to the difference in depth between A and B is calculated
- in order to calculate the pore water pressure, the phreatic surface ($u = \gamma_w \cdot h$) height is assumed as the mean value between h_1 and h_2

The value determined is still slightly conservative with respect to the real value.
For calculation purposes of the pore water pressure, the parameter Ru (pore pressure parameter) related to the soils and not to the water tables can be used.
The parameter Ru allows the user to calculate the excess of pore water pressure (Δu) due to shear stresses according to the following equation:

$$\Delta u = W\, Ru/dx$$

being W the slice total weight and dx its width.
The exceeding pressure Δu is added to the pressure value u obtained in defining the phreatic surface, therefore the pressure Δu can be added to or replace the pressure value u.

Calculation of the forces acting on the soil free surface
When a phreatic surface is located on the soil's free profile, a hydrostatic pressure (u_t) develops which is then calculated for each slice by the following equation:

$$u_t = \gamma_w \cdot h_t$$

where h_t is the height of the water column above the soil's free profile

Calculation of the filtration forces
In case of inclined phreatic surface, a dragging force parallel to the flow direction generates within the soil.
The vertical component of this force, heading downhill, is implicitly calculated when the pore water pressure at the base is determined. In fact, the use of a value of the height of the phreatic surface

lower than the hydrostatic pressure produces a lower uplift, due to the dragging effect towards the bottom produced by the filtration forces (for a phreatic surface decreasing downhill).
The horizontal component (figure 18) is calculated on the base of the equilibrium of the hydrostatic pressures acting on the right margin of the slice (force directed leftward), on the left margin (force directed rightward) and on the bottom (force usually directed leftward). In case of a horizontal phreatic surface, the sum of these forces is nil, whereas if the phreatic surface is inclined, the sum of the above forces has a positive value and the resulting force has a leftward direction (for a phreatic surface which reduces downhill).

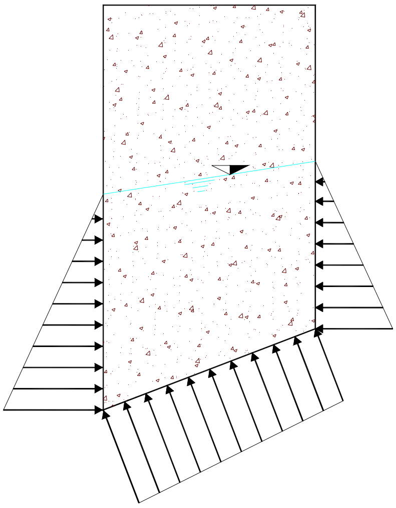


Figure 18

Calculation of the hydrostatic pressure

When a phreatic surface terminates within the soil, for example against an impermeable surface, a hydrostatic pressure generates within the soil. The calculation of this pressure is obtained by the procedure described in the previous paragraph.

2.6 OVERALL STABILITY ANALYSIS

The overall stability analysis is a limit equilibrium stability.
The user must define:

- the number of surfaces to generate
- the calculation method to adopt (Bishop, Janbu)
- the type of surfaces (circular or random polygonal)
- the initiation point downhill of the failure surfaces (usually from 0.5 to 1.0 times the height of the retaining work)
- the termination point uphill the failure surfaces (usually from 1.5 to 2.0 times the height of the retaining work)
- the minimum length of the segments which make up each sliding surface
- a minimum depth under which the failure surfaces cannot extend
- limit to the angle with which the first segment of the sliding surface is generated
- the calculation method: rigid or displacement with the relevant calculation parameters

Among the safety coefficients determined (one for each surface) the lowest is the slope safety coefficient.
According to the calculation method adopted for the reinforcing units, the maximum tensile stress acting on the reinforcing units is provided (see paragraph 2.5.1).

2.7 INTERNAL STABILITY ANALYSIS for reinforced soil structures

The internal stability analysis is a stability check against limit equilibrium.
The user must define:

- the retaining work or blocks to be checked
- the number of surfaces to generate
- the calculation method to adopt (Bishop, Janbu)

- the type of surfaces (circular or random polygonal)
- the termination point uphill the failure surfaces (usually from 1.5 to 2.0 times the height of the retaining work)
- the minimum length of the segments which form each sliding surface
- a minimum depth under which the failure surfaces cannot extend
- limit of the angle with which the first segment of the sliding surface is generated
- the calculation method: rigid or displacement with the relevant calculation parameters

Among the safety coefficients determined (one for each surface) the lowest is the slope safety coefficient. According to the calculation method adopted for the reinforcing units, the maximum tensile stress acting on the reinforcing units is provided (see paragraph 2.5.1).

2.8 STABILITY ANALYSIS OF THE REINFORCED SOIL AS A RETAINING WALL

This type of analysis is conducted by using a procedure which consists of the following steps:

1) Selection of the structure to check (user's choice)
2) Geometrical check of the user's choice
3) Definition of the retaining wall (backfill profile)
4) Calculation of the stabilising forces
5) Calculation of the maximum thrust
6) Check against sliding
7) Check against overturning
8) Check against bearing capacity

2.8.1 Selection of the portion of a reinforced soil structure to be considered a retaining wall

The user must select the block to check as a retaining structure. All blocks of the retaining structure which are located over the selected block become part of the retaining wall.

2.8.2 Geometric check of the chosen portion

Macstars W preliminarily checks if the user's choice generates a retaining wall according to the conditions detailed in paragraph 2.3.3.
If the user's choice does not meet the above conditions, the program will display a message showing an inclination less than 70°, not giving the possibility to proceed with the calculation.

2.8.3 Definition of the retaining wall

The program automatically defines the wall's structure, considering all selected blocks and the soil portions above them (figure 19Figura_19, dashed line).
This selection is fundamental since everything within this profile forms the retaining wall and therefore all relevant weights are used when considering sliding and overturning as stabilising load

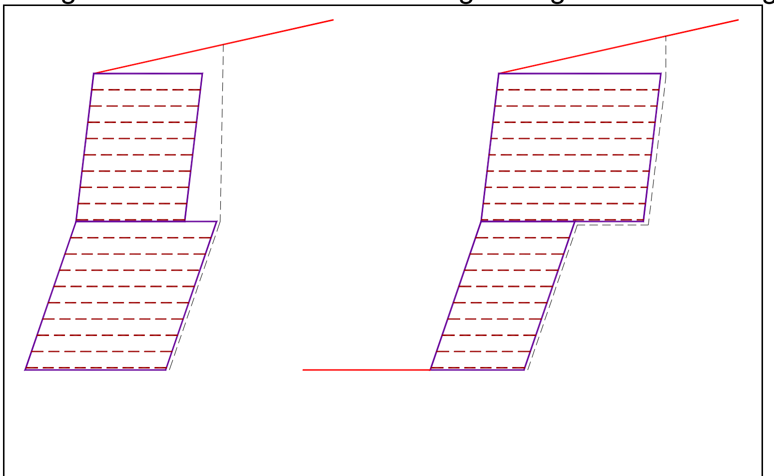


Figure 19

2.8.4 Calculation of the forces acting within the retaining wall

The calculation procedure which allows to determine the forces acting within the retaining wall (forces and moments) is based upon the data coming from the program section which conducts the limit equilibrium stability checks.

The retaining wall, considered as a fictitious single sliding surface, is divided into slices and for each slice the following data are used:

1) total weight
2) forces due to distributed surcharge loads
3) forces due to linear loads (without transversal distribution)
4) forces due to repeated or isolated point loads (without transversal distribution)
5) forces due to tiebacks (without transversal distribution)
6) forces on the free surface due to the presence of a phreatic surface
7) pore water pressure at the base
8) forces due to seismic actions
9) internal forces due to the level variations of the water table (filtration or hydrostatic thrust)

Then the program calculates the overall stabilising force acting along the base, the unstabilising horizontal force, the stabilising moment and the overturning moment.

Overall stabilising forces

The following procedure is employed:

a) calculation of the vertical force acting on the base (F_v)
b) calculation of the horizontal stabilising force (F_h) due to the forces from 2 to 6
c) calculation of the uplift due to the pore water pressures at the base (U)
d) calculation of the total force acting on the base $N = F_v – U$
e) calculation of the resisting force due to the cohesion (F_{coes}) on the base
f) Calculation of the mean internal friction angle (ϕ_{med}) on the base
g) Calculation of the total stabilizing resistant force (F_{stab})

$$F_{stab} = N \,.\, \tan(\phi_{med}) + F_{coes} + F_h$$

Total unstabilising forces

The total unstabilising force (horizontal) acting inside the retaining wall (F_{hun}) is obtained by adding forces 8 and 9.

Total stabilising moment

The total stabilising moment (M_s) is obtained by adding all contributions due to the single moments of the forces from 1 to 6 with respect to the wall's downhill edge.

Total overturning moment

The total overturning moment (M_o) is obtained by adding all contributions due to the single moments of the forces from 8 to 9 with respect to the wall's downhill edge

The un-stabilizing moment (M_u), due to the interstitial pressures at the base, is considered as well.

2.8.5 Calculation of the maximum forces acting on the wall

The calculation of the forces acting on the wall, due to the thrust generated by the backfill, is carried out with a procedure based on the data coming from the program section which conducts the limit equilibrium stability checks.

The adopted procedure is the following:

1) the program analyzes 200 sliding surfaces which include the entire wall base and terminate uphill by random directions or directions obtained by the Rankine + Mononobe e Okabe equation
2) the program analyzes each single surface in order to determine the thrust applied to the retaining wall and the relevant overturning moment
3) the soil portion within a surface is divided into slices and for each slice all the forces described in the previous paragraph relevant to the retaining wall are calculated, by deducting all the forces already considered for the retaining wall and by adding all forces due to the reinforcements intersected by the surface (using the rigid model); the forces thus obtained are those which generate the thrust acting on the retaining wall

4) the thrust acting on the wall is determined by adding all the slices contributions
5) the thrust due to a single slice is obtained by solving the forces polygon consisting of four forces: the resultant of the horizontal components, the resultant of the vertical components, the reaction at the base of the slice inclined by the friction angle with respect to the base, the active thrust assumed as acting horizontally (this assumption complies with Bishop assumptions in the stability analysis)
6) the overturning moment due to the thrust is obtained by considering the single contributions of all forces with respect to the retaining wall's edge downhill
7) the value of the thrust (S_a) for the stability check of the retaining wall is obtained by considering the maximum of the thrusts calculated on all surfaces; the relevant moment (M_a) is used for the stability checks against overturning.

2.8.6 Stability checks against sliding

The safety coefficient against sliding (F_{ss}) is given by the following equation:

$$F_{ss} = F_{stab}/F_{htot}$$

Where $F_{htot} = (S_a + F_{hun})$ = total horizontal force

F_{stab} = total stabilising force acting at the base of the wall

S_a = maximum active thrust acting on the wall

F_{hun} = horizontal un-stabilising force acting on the wall (due to seismic actions or hydraulic forces)

2.8.7 Check against overturning

The safety coefficient against overturning (F_{ov}) is given by the following formula:

$$F_{ov} = (M_{stab} - M_u) / (M_a + M_{ov})$$

where summarizing:

M_{stab} = stabilising moment due to the forces acting on the wall

M_u = overturning moment due to the interstitial forces acting at the base of the wall

M_a = overturning moment due to the maximum active thrust acting on the wall

M_{ov} = overturning moment due to the un-stabilising horizontal forces acting on the wall (due to seismic actions or hydraulic forces)

2.8.8 Check against the bearing capacity

The stability check against the wall foundation bearing capacity can be conducted both by assigning the ultimate pressure of the foundation soil (p_u), and having the program calculate this value, as described in paragraph 2.8.9.

The procedure to check the wall foundation bearing capacity consists of the following steps:

1) the program determines the value of the eccentricity (e) from the relationship

$$e = B / 2 - [(M_{stab} - M_u) - (M_a + M_{ov})] / N$$

2) the program determines the reduced width (B_r) of the foundation's base

$$B_r = B \qquad e < 0$$
$$B_r = B - 2e \qquad e > 0$$

3) in case of ultimate pressure provided by the user, the program determines the mean equivalent pressure (p_{meq}) by the equation

$$p_{meq} = N / B_r$$

4) in case of ultimate pressure calculated by the program, the mean equivalent pressure (p_{meq}) is determined by the equation

$$p_{meq} = R / B_r$$

where R= the inclined vectorial resultant of the vertical load (N) and the total horizontal force acting on the base (F_{htot})

5) the program determines the safety coefficient of the bearing capacity (F_{scp}) by using the following equation

$$F_{scp} = p_u / p_{meq}$$

where p_u =ultimate pressure of the foundation soil provided by the user (considered in this case with a vertical direction) or calculated by the program (considered inclined as R)

2.8.9 Calculation of the ultimate pressure

The ultimate pressure of the foundation soil is calculated by using a general method which refers to the classical method of calculation of the limit equilibrium (Terzaghi, Hansen Meyerhof), and which allows the user to take into consideration complex stratigraphic or geometrical situations.
The procedure adopted, with reference to figure 20, is the following:

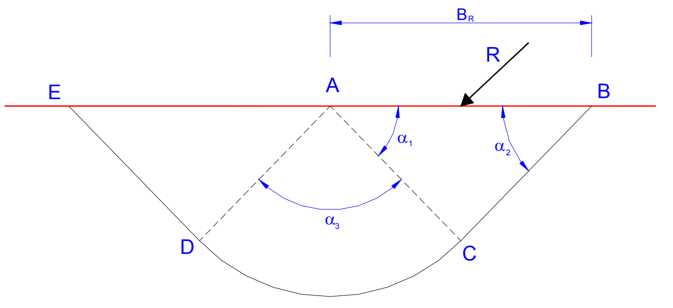


Figure 20

1) the program considers a foundation with a width Br subject to an inclined load R (see paragraph 2.8.8), which extends to the infinite, in the third direction
2) then 225 surfaces identified as straight lines (BC) – spirals (CD) – straight line (DE) are defined; for each surface point C is obtained by intersecting the lines exiting from A and B with the angles α_1 and α_2 (which can vary between 10° and 70° at intervals of 4°), whereas the segment CD is a logarithmic spiral with an angle α_3 = 90°, tangent in C to the segment BC; the segment DE instead is tangent in D to the same logarithmic spiral; some geometrical checks allow to eliminate those possible surfaces non- compatible with the problem's geometry.
3) For each defined surface a limit equilibrium stability analysis is conducted using Janbu method by increasing the acting pressure from the initial value (R/B_r) up to a value (p_1) which provides Fs=1.0
4) The smallest of all p_1 values calculated for all the generated surfaces corresponds to the ultimate pressure of the wall's foundation soil

2.9 ANALYSIS FOR SUPPORT STRUCTURE (GABION STRUCTURE)

The analysis in question is performed with a procedure divided into the following steps:
1) Selection of wall to be analysed
2) Definition of wall
3) Calculation of stabilising forces
4) Calculation of maximum thrust
5) Sliding analysis
6) Overturning analysis
7) Load-bearing analysis

2.9.1 Selection of wall to be analysed

The user selects the wall to be analysed as a support structure.

2.9.2 Definition of wall

The program automatically defines the wall structure, considering the gabion wall selected and those parts of the soil vertically above it (**fig. 21**, yellow line).
This selection is important, as everything found inside this profile constitutes the retaining wall and, therefore, all the relative weights are stabilising with respect both to sliding and overturning.

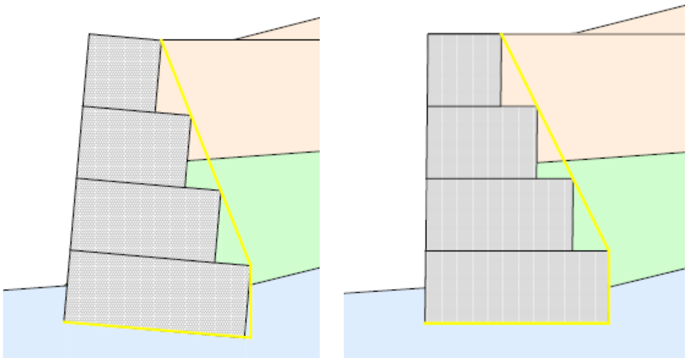


Figure 21

2.9.3 Calculation of the actions inside the wall

The procedure which allows calculation of the actions inside the wall (forces and moments) is based on the data derived from the section of the program which analyses the limit equilibrium stability.
The wall, considered to be a fictitious single sliding surface, is subdivided into segments and the following values are used for each segment:

1) total weight
2) forces due to distributed loads
3) forces due to linear loads (without transversal spread in this analysis)
4) forces due to repeated or single spot loads (without transversal spread in this analysis)
5) forces due to ties (without transversal spread in this analysis)
6) forces on free boundary due to presence of water tables
7) pore pressure at the base
8) forces due to seismic loads
9) internal forces due to variation in water table level (filtration or hydrostatic thrust)

The program calculates the total stabilising force along the base, the unstabilising horizontal force, the stabilising moment and the overturning moment.

Total stabilising force

The following procedure is applied:

a) calculation of the total vertical force acting on the base ($\mathbf{F_v}$)
b) calculation of the stabilising horizontal force ($\mathbf{F_h}$) due to the forces from 2 to 6
c) calculation of the resultant force (under-thrust) of the pore pressures at the base (**U**)
d) calculation of the total effective force acting on the base $\mathbf{N = F_v - U}$
e) calculation of the resistant force due to the cohesion ($\mathbf{F_{coes}}$) on the base
f) calculation of the average internal angle of friction (ϕ_{med}) on the base
g) calculation of the total stabilising resistant force ($\mathbf{F_{stab}}$)

$$\mathbf{F_{stab} = N \cdot \tan(\phi_{med}) + F_{coes} + F_h}$$

Total unstabilising force

The total unstabilising force (horizontal) inside the retaining wall ($\mathbf{F_{hin}}$) is obtained by adding together forces 8 and 9.

Total stabilising moment

The total stabilising moment ($\mathbf{M_s}$) is obtained by adding together the contributions due to the single moments of the forces from 1 to 6 with respect to the downhill edge of the wall.

Total overturning moment

The total overturning moment (**M_r**) is obtained by adding together the contributions due to the single moments of the forces from 8 to 9 with respect to the downhill edge of the wall.
The unstabilising moment (**M_u**) due to the pore pressures at the base is also considered.

2.9.4 Calculation of the maximum forces acting on the wall

The calculation of the forces acting on the wall, due to the thrust of the soil behind it, is obtained by a procedure based again on the data derived from the section of the program which analyses the limit equilibrium stability.
The following procedure is used:

- 200 fictitious sliding surfaces are analysed which cover the entire base of the wall and end uphill according to random directions or directions given by the Rankine + Mononobe and Okabe formulae
- each surface is analysed to determine the thrust applied to the wall and the relative overturning moment
- the portion of the soil inside a surface is subdivided into segments and all the forces already seen in the previous section relative to the retaining wall are determined for each segment, subtracting all the forces already considered in the wall; the forces thereby obtained are those from which the thrust on the wall is derived
- the thrust on the wall is calculated by adding together the contributions of the single segments
- the thrust due to the single segment is obtained by resolving the polygon of forces consisting of four total forces: the resultant of the horizontal components, the resultant of the vertical components, the reaction at the base of the inclined segment of the angle of friction with respect to the base, the active thrust assumed in a horizontal direction (assumed to be in compliance with the Bishop assumption in the stability analyses).

  In the particular case in which the strength characteristics of the soil are uniform and the soil thrust may be compared with that of a wedge, where the direction of the friction forces at the base of the wedge and between the wall and the wedge are known, it is easy to calculate the intensity of these forces (in the assumption of a rigid wedge) simply by using the global equilibrium equations for the forces.

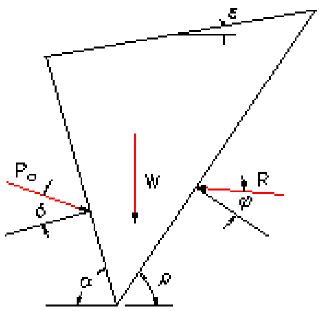


Figure 22

In the case in which the sliding surface is generic and the soil characteristics are not uniform, the thrust calculation requires a method which is generalizable but which gives the same solution known in the case of a simple wedge.
It was decided to use the limit equilibrium method to determine the thrust, adapting the calculation procedure as shown below.
By examining the simple case below:

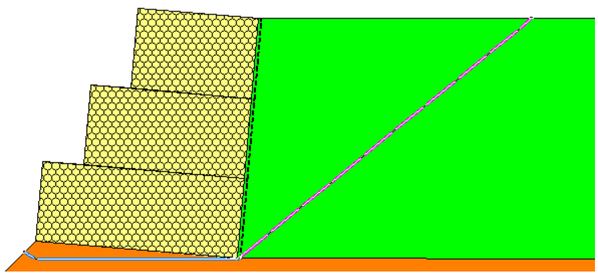


Figure 23

the following diagram is obtained:

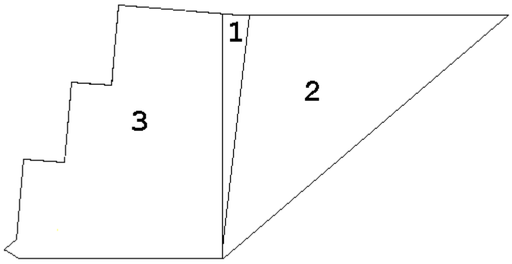


Figure 24

The thrusts to be calculated are on the interface between wedge **1** and **2**. Since it is not possible to directly apply the limit equilibrium method to the wedge **2**, the forces are calculated by the limit equilibrium method at wedges **1**, **1+2**, which may be subdivided into vertical strips as required by the method.
The unknown to be calculated is the force **F** between the wedge **2** and the gabion **1+3**, this force is inclined by the angle of friction δ with respect to the normal at the contact surface between the two wedges.

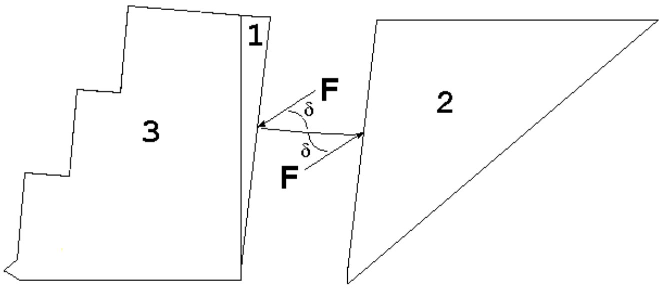


Figure 25

In order to calculate the force **F ,** the system of the wedges **1** and **1+2** is resolved, by varying the angle δ until the forces **N1** and **N2** are equal: see diagram:

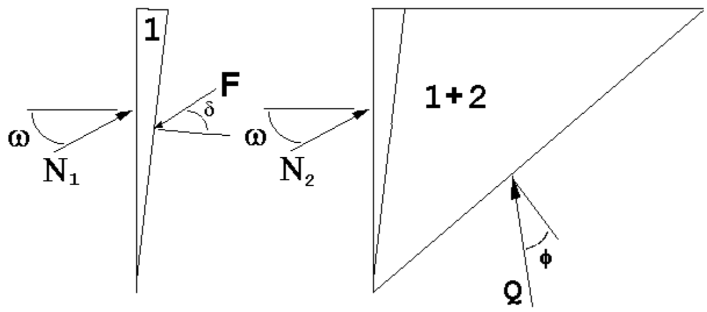


Figure 26

The procedure is as follows:
In wedge **1** where the two directions of the forces **F** and **N1** are known, their intensity is calculated by closing the polygon of the forces acting on them. Assuming, for simplicity, that only the weight **W** acts on the wedge, the intensity of the forces is calculated from the following polygon:

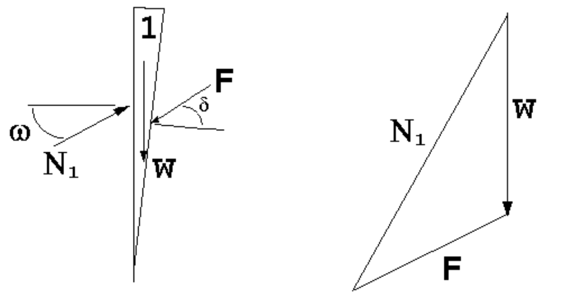


Figure 27

Similarly, on wedge **1+2** , knowing the weight and the directions of forces **N2** and **Q** (inclined by the angle of friction ϕ with respect to the normal at the base of the wedge) the intensities are calculated.

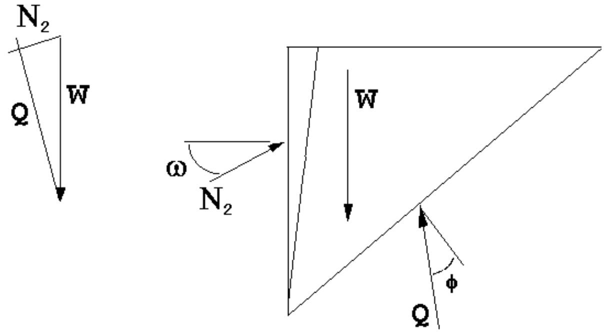


Figure 28

The algorithm implemented in the program allows angle ω to be varied until **N1** = **N2** is obtained.

It may therefore be considered that the equilibrium of wedge **2** is obtained as the difference of wedge **1+2** and wedge **1**; with this operation wedge **1** is annulled, as are forces **N1** and **N2**, so the force **F** is therefore calculated.

Knowing force **F**, it is possible to calculate the equilibrium of gabion **1+3** as the equilibrium of a rigid body.

The subdivision into segments of wedges **1** and **1+2** does not alter the calculation procedure as it is considered that the forces on the vertical faces of the segment act along the constant direction ω from which the forces **N1** and **N2** are derived as the sum of the contributions of the single segments. The weights of the single segments, the external forces and the thrust at the base of the segments are, on the other hand, variables, linked to the geometry of the sliding surface, the properties of the materials and the load conditions.

The equilibrium of the generic segment is therefore as shown in the following diagram:

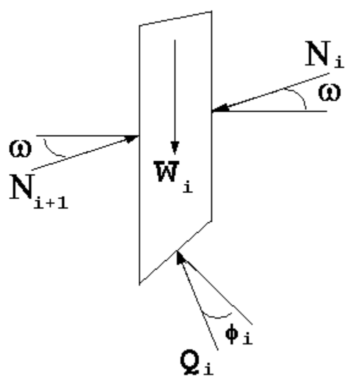


Figure 29

where the only unknown is the intensity of **Ni+1**.

In the case of stepped gabions on the uphill side (see Figure)

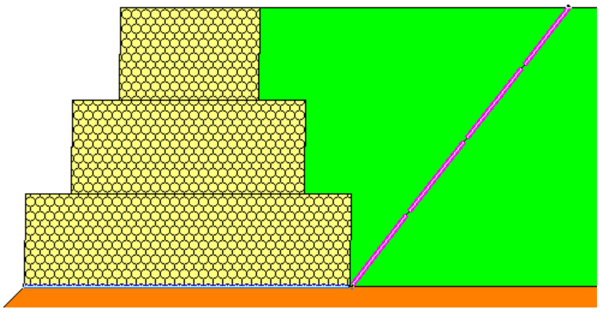


Figure 30

The following equivalent diagram is obtained:

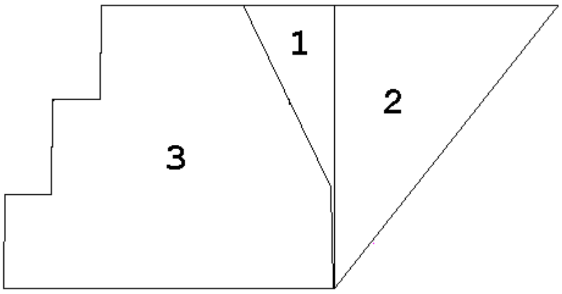


Figure 31

in which the unknown is the force of the wedges **1+2** on the gabion **3**; the direction of this force is known to be normal to the average inclination of the wedge **1** and rotated by the angle of friction δ between the wedge **1** and the gabion **3**.

As already mentioned, the inter-segment forces between wedges **1** and **2** should be noted; these forces may be calculated (as already explained) if their direction is fixed.

Also in this case, the algorithm is based on the calculation of the angle ω which allows equilibrium between wedges **1** and **2**, i.e. the angle ω is sought for which the forces **N1** and **N2** are balanced.

See figure below.

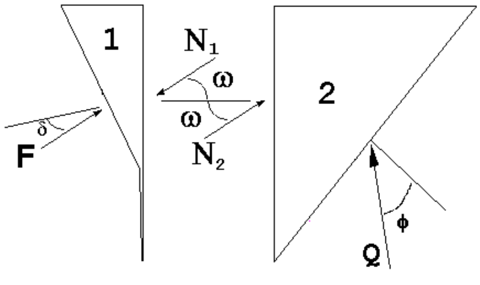


Figure 32

In this case, the sum of the wedges **1** and **2** annuls the force **N1** with **N2** so **F** is the unknown sought.

— the overturning moment due to the thrust is obtained by considering the single contributions of all the forces with respect to the downhill edge of the wall.

— the thrust value (**S_a**) for the stability analysis of the retaining wall is obtained by considering the maximum of the thrusts calculated on all the surfaces; the relative moment (**M_a**) is in turn used in the overturning analysis

2.9.5 Sliding analysis

The siding factor of safety (**F_{ss}**) is given by the following relationship:

$$F_{ss} = F_{stab} / F_{htot}$$

where: $F_{htot} = (S_a + F_{hin})$

and where:
F_{stab} = total stabilising force acting at the base of the wall
S_a = maximum thrust (active) acting on the wall
F_{hin} = unstabilising horizontal force acting on the wall (due to earthquake or hydraulic forces)

2.9.6 Overturning analysis

The overturning factor of safety (**F_{sr}**) is given by the following relationship:

$$F_{sr} = (M_{stab} - M_u) / (M_a + M_r)$$

and where:
M_{stab} = stabilising moment due to the forces acting on the wall
M_u = overturning moment due to the pore pressure forces at the base of the wall
M_a = overturning moment due to the maximum thrust (active) acting on the wall
M_r = overturning moment due to the unstabilising horizontal forces acting in the wall (due to earthquake or hydraulic forces)

2.9.7 Load-bearing analysis

The analysis of the load-bearing capacity of the wall foundation may be carried out both by assigning the ultimate pressure of the foundation soils (**p_u**), and by calculating this value in the program itself, as described under point 2.8.9 below.
The procedure for analysing the load-bearing capacity at the base of the wall is divided into the following steps:

1) the eccentricity value (**e**) is calculated by the formula
$$e = B / 2 - [(M_{stab} - M_u) - (M_a + M_r)] / N$$
2) the reduced length (**B_r**) of the foundation base is calculated
$$B_r = B \qquad e < 0$$
$$B_r = B - 2 . e \qquad e > 0$$
3) in the case of ultimate pressure assigned by the user, the average equivalent pressure (**p_{meq}**) is calculated as follows:
$$p_{meq} = N / B_r$$
4) in the case of ultimate pressure calculated by the program, the average equivalent pressure is calculated as follows:
$$p_{meq} = R / B_r$$
where **R** = inclined vectorial resultant of the vertical load (**N**) and the total horizontal force acting on the base (**F_{htot}**)
5) the load-bearing safety factor (**F_{scp}**) is calculated as follows
$$F_{scp} = (p_u) / p_{meq}$$
where **p_u** = ultimate pressure of the foundations soils assigned by the user (in a vertical direction) or calculated by the program (inclined as per **R**)

2.9.8 Calculation of ultimate pressure

The ultimate pressure of the foundation soils is calculated with a general method, based on the classic limit equilibrium methods (Terzaghi, Hansen, Meyerhof), and which allows complex stratigraphy or geometrical situations to be taken into consideration.
The procedure used, with reference to **Fig. 33**, is as follows:

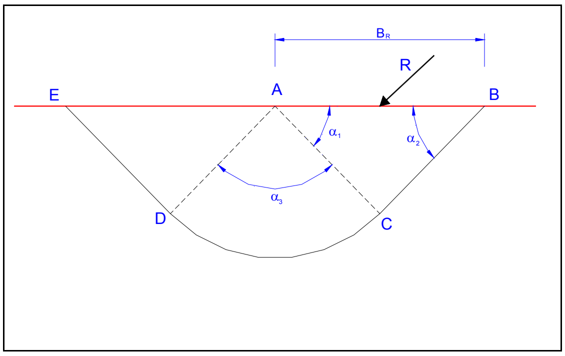


Figure 33

1) a foundation is considered with width **B**$_r$ subject to inclined load **R** (ref. point 2.8.8), considered ribbon-like, i.e. infinitely extended in the third direction
2) 225 surfaces of the straight (**BC**) – spiral (**CD**) – straight (**DE**) type are defined; point **C** is obtained for each surface by intersecting the half-straight lines leading from **A** and **B** with angles α_1 and α_2 (variable between 10° and 70° at intervals of 4°), whilst section **CD** is of the logarithmic spiral type with angle α_3 = 90° , tangent at **C** to the section **BC**; lastly, section **DE** is tangent at **D** to the same logarithmic spiral; some geometrical controls allow rejection of any surfaces not compatible with the geometry of the problem
3) a limit equilibrium stability analysis is carried out for every surface defined, using the Janbu method, increasing the acting pressure from the initial value (**R/B**$_r$) up to a value (**p**$_1$) so as to obtain **Fs=1.0**
4) the smallest of all the values **p**$_1$ calculated for all the surfaces generated is the ultimate pressure of the wall's foundation soils

2.10 SLIDING CHECK FOR SOIL REINFORCED STRUCTURES OR BLOCKS

The stability analysis against sliding of a soil reinforced structure (or for a single block) is conducted by using a procedure similar to the one used for the wall checks.
The main steps of this type of analysis are:

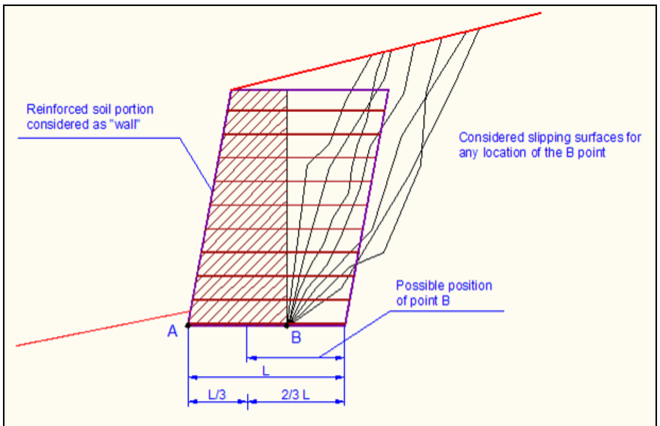


Figure 21

1) selection of the work/block to check
2) the program considers that the base resistant to sliding is included between the downhill corner edge of the block (figure 21Figura_21, point A) and a point located in the uphill line with an extension equal to half the base of the block (figure 21Figura_21, point B).
3) then 200 potential sliding surfaces are generated changing the position of point B and the initiation point of the same point (100 surfaces are generated at random and a 100 by Rankine's theory)
4) for each surface a retaining wall is considered defined uphill by the vertical profile exiting from B towards the free surface.
5) then the surface which gives the maximum thrust on the wall is determined and for this surface the resistant force acting on the base is determined. Then the program determines the safety coefficient against sliding by using the same data used for the retaining wall and the same

parameters of resistance provided by the user in relation to the type of contact determined at the base

The reinforcements which can be possibly intersected by the uphill segment of the sliding surface are kept into consideration in the calculation of the thrust (thrust reduction effect) and the tensile stress acting on the reinforcing element is obtained by the rigid model.

2.11 INTERNAL STABILITY ANALYSIS (ANALYSIS OF THE SLIDING AND OVERTURNING BETWEEN A LAYER OF GABIONS AND A LAYER BELOW)

The internal stability analysis of a gabion retaining structure is the analysis under sliding and overturning of a layer from which it is formed with respect to the layers below, and it uses a similar procedure to that used in the analysis of the entire structure.
The basic steps of the analysis are as follows:
- selection of the gabion layer subject to analysis.

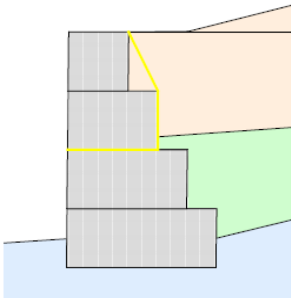


Figure 35

- 200 potential sliding surfaces are generated (100 surfaces in a random manner and 100 surfaces according to the Rankine + Mononobe Okabe theory)
- a retaining wall type arrangement is adopted for every surface, in which the wall is defined uphill of the yellow line shown in Figure 35.
- the surface which provides the maximum thrust on the wall is determined and the resistant force at the base is calculated for this. The sliding and overturning safety factor is then calculated, using the same formulae seen for the retaining wall.

2.12 SETTLEMENT CALCULATION

This procedure, which allows to calculate the soil settlements is based on the data provided by the section of the program which conducts the limit equilibrium stability checks and consists of the following phases:
1) Calculation of the loads
2) Calculation of the change in the induced tensional state
3) Calculation of the soil settlements

2.12.1 Calculation of the loaded area

The soil profile which is added to the pre-existing soils (defined by the user) is considered as a fictitious sliding surface and the soil delimited by this surface is divided into slices (maximum width 2 meters) and for each slice the following values are used:
1) total weight
2) forces due to the distributed loads
3) pore water pressure at the base

These three values allow the software to calculate for each slice the vertical pressure acting on the soil layers below, that is the strip load acting on the slice. Therefore the program determines a number of strip loads, applied at different levels.

2.12.2 Calculation of the induced stresses

The vertical line where settlements are calculated is defined by the user and is considered in the problem’s geometry, thus obtaining the corresponding stratigraphy by defining for each layer the required input data.

Each layer is additionally divided into elementary segments and for each segment the change in the induced tensional state is calculated by superimposing the effects of the single strip loads.
For the calculation of the change in the induced tensional state, the program refers to Jumikis (1971) equations which are based on the following hypothesis:

a) the foundation soil is considered as a linear-elastic, homogeneous and isotropic semi-space
b) the area affected by the load action is located at the upper limit of the semispace
c) the load area is assumed to be infinitely flexible

The equations used are:

$$\Delta\sigma_z = q / \pi \,.\, [\tan^{-1}((x + D) / z) - \tan^{-1}((x - D) / z) - z \,.\, (x - D) / ((x-D)^2+z^2) + z \,.\, (x + D) / ((x-D)^2+z^2)]$$
$$\Delta\sigma_x = q / \pi \,.\, [\tan^{-1}((x + D) / z) - \tan^{-1}((x - D) / z) + z \,.\, (x - D) / ((x-D)^2+z^2) - z \,.\, (x + D) / ((x+D)^2+z^2)]$$
$$\Delta\sigma_y = \nu \,.\, (\Delta\sigma_z + \Delta\sigma_x)$$

where: $\Delta\sigma_z$, $\Delta\sigma_x$, $\Delta\sigma_y$ =variations of the vertical (z) and horizontal (x,y) tensional state in the i-th soil segment
q = applied load
ν = Poisson coefficient (assumed equal to 0.30)
D = semi-width of the load
x,z = coordinates of the point of calculation of the tensional state relevant to a reference system which originates in the axis of the load

2.12.3 Calculation of the settlements

Once the profile of the induced tensional state variation has been determined, the calculation of the settlement is conducted by applying the elasticity equations to each single soil segment
The calculation of the settlement is extended into the soil depth until the vertical tensional variation induced by the load ($\Delta\sigma_z$) is lower by 10% of the initial geo-static stress.
The calculation takes into consideration incoherent and coherent soils.

Granular soils
The calculation of the elementary failure relevant to the i-th segment (s_i) is carried out by using the equation: $s_i = [\Delta\sigma_z - \nu_i (\Delta\sigma_x + \Delta\sigma_y)] \,.\, \Delta h_i / E_i$

where: $\Delta\sigma_z$, $\Delta\sigma_x$, $\Delta\sigma_y$ = variations of the vertical (z) and horizontal (x,y) tensional state in the i-th soil segment
ν_i = Poisson coefficient (assumed equal to 0.30) in the i-th soil segment
E_i = mean deformability modulus in the i-th soil segment
Δh_i = thickness of the i-th soil segment

The parameters of elasticity ν and E are directly provided by the user during the soil input data procedure

Cohesive soils
The calculation of the elementary settlement relevant to the i-th segment (s_i) is carried out by using the equation:

$$s_i = \mu_i \,.\, s_{ed,i}$$

where:

$s_{ed,i}$ = edometric settlement of the i-th soil segment
$\beta_i = a + \alpha\,(1\text{-}A)$ = corrective factor of the i-th soil segment
A = Skempton's parameter of the pore water pressure
α = adimensional coefficient related to the geometry

The parameters A and α are provided by the user

The edometric settlement $s_{ed,i}$ is calculated by making a distinction between over-consolidated soils (OC) and normal-consolidated soils (NC)

a) NC soils

$$s_{ed,i} = C_C \cdot \log_{10} (\sigma'_c \ / \ \sigma'_o) \cdot \Delta h_i$$

where:

σ'_o	effective geo-static vertical pressure
$\sigma'_f = \sigma'_o + \Delta\sigma_z$	effective final vertical pressure
Δh_i	thickness of the i-th soil segment
C_C	coefficient of primary compression (provided by the user)

b) OC soils

There are two possible cases:

$\sigma'_f > \sigma'_c$ $\quad s_{ed,i} = [C_R \cdot \log_{10} (\sigma'_c \ / \ \sigma'_o) + C_C \cdot \log_{10} (\sigma'_f \ / \ \sigma'_c)] \cdot \Delta h_i$

$\sigma'_f < \sigma'_c$ $\quad s_{ed,i} = C_R \cdot \log_{10} (\sigma'_f \ / \ \sigma'_o) \cdot \Delta h_i$

where:

σ'_o	effective geo-static vertical pressure
σ'_c	pre-consolidation pressure (provided by the user at the bottom and the top of each cohesive layer)
$\sigma'_f = \sigma'_o + \Delta\sigma_z$	effective final vertical pressure
Δh_i	thickness of the i-th soil segment
C_C	coefficient of primary compression (provided by the user)
C_R	coefficient of re-compression (provided by the user)

2.13 CALCULATION OF THE YIELD ACCELERATION

Newmark developed a displacement method in 1965 based on the possibility of predicting the deformation of a slope subjected to seismic action. Newmark illustrated the analogy between a potentially unstable slope and a rigid block in equilibrium on an inclined plane.

100

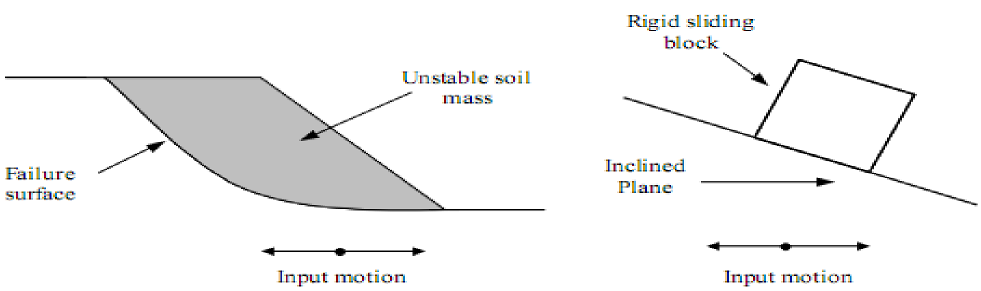


Figure 5.1 Analogy between potentially unstable slope and a rigid block on an inclined plane

Figure 36

The sliding mass is considered in the Newmark method as a rigid body which slides in a downhill direction.

2.13.1 Application requirements

The forecasting capacity of the method obviously depends on the consistency of the actual behaviour with the design layout.

The deformations inside the soil subject to analysis must be negligible with respect to the overall movement.

The assumption of a rigid block implies that the mobilisation on the sliding surface is uniform.

The behaviour of the material on the sliding surface may be likened to a rigid-plastic behaviour. There are no significant variations in the pore pressure induced by an earthquake.

2.13.2 Yield Acceleration

From the analysis of the rigid block according to the Newmark theory, it is seen that for a relative movement between block and support surface to occur, a certain level of acceleration must be exceeded.

The yield acceleration is normally considered to be the pseudo-static acceleration value which makes the rigid block unstable. This occurs in the soil when the level of acceleration of the slope makes the factor of safety 1.

As shown for the first time by Richards and Elms (in Seismic behaviour of gravity retaining walls, JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION April 1979), Newmark's Displacement Method is applicable for gravity walls which do not collapse structurally but lose their function due to the displacement of the works.
The movement of the walls does not take place in a continuous manner, but in small increments during the earthquake shocks. When the acceleration is in a downhill direction, the inertia forces are in an uphill direction, creating a passive thrust situation. Since the passive thrust is very high, the soil and the wall move solidly.
When the acceleration is in an uphill direction, the risk of failure due to active thrust is increased.
At the acceleration peaks during an earthquake, when the yield acceleration is exceeded, the wall undergoes rapid accelerations and small displacements in a downhill direction.
In consideration of the fact that the displacement method is substantially subdivided into two parts - calculation of the yield acceleration and calculation of the displacement - and considering that there are numerous stability analyses, whilst the Newmark method does not require information on the method of calculation of the yield acceleration (so it may be applied on the most critical analysis) it has been preferred to keep the two parts of the calculation procedure separate.
A function has been implemented in the MacStars W program for calculation of the yield acceleration, which is applicable both for the stability of retaining structures, using either reinforced soil or gabions, and for slopes. This function allows calculation of the acceleration value which results in a sliding safety factor equal to 1, with activation of all the possible sliding surfaces.

For calculation of the displacement, use may be made of the USG (U.S. Geological Survey) program or the Newmark program, developed by Maccaferri, and conceived in particular for the Italian regulations.

Table of contents

1. USER'S MANUAL (Please see User's manual) 2
2. REFERENCE MANUAL 2
2.1 INTRODUCTION 2
2.2 BASIC DEFINITIONS (see fig.1) 2
2.3 TYPES OF STABILITY ANALYSIS (CHECKS) 3
2.3.1 Overall stability check 3
2.3.2 Internal stability check 3
2.3.3. Stability check of the structure as a retaining wall (Sliding, Overturning and Foundation load-bearing capacity) 4
2.3.4 Check against sliding 5
2.3.5 Internal sliding analysis (analysis of sliding between a layer of gabions and the layer above) 5
2.3.6 Check against soil settlements 5
2.3.7 Stability Analysis checks for a given surface. 5
2.4 BEHAVIOUR OF REINFORCING UNITS (reinforced soils) 5
2.4.1 Rigid model 5
2.4.2. Deformative model (Displacement Method) 7
2.5 STABILITY ANALYSIS CONDUCTED USING THE LIMIT EQUILIBRIUM METHOD 8
2.5.1 Methods used by MACSTARS W 9
2.5.2 Generation of the failure surfaces 10
2.5.3 Sub-division into slices 11
2.5.4 Surcharge loads 11
2.5.5. Soil reinforcement 14
2.5.6 Water Tables 15
2.6 OVERALL STABILITY ANALYSIS 17
2.7 INTERNAL STABILITY ANALYSIS for reinforced soil structures 17
2.8 STABILITY ANALYSIS OF THE REINFORCED SOIL AS A RETAINING WALL 18
2.8.1 Selection of the portion of a reinforced soil structure to be considered a retaining wall 18
2.8.2 Geometric check of the chosen portion 18
2.8.3 Definition of the retaining wall 18
2.8.4 Calculation of the forces acting within the retaining wall 19
2.8.5 Calculation of the maximum forces acting on the wall 19
2.8.6 Stability checks against sliding 20
2.8.7 Check against overturning 20
2.8.8 Check against the bearing capacity 20
2.8.9 Calculation of the ultimate pressure 21
2.9 ANALYSIS FOR SUPPORT STRUCTURE (GABION STRUCTURE) 21
2.9.1 Selection of wall to be analysed 21
2.9.2 Definition of wall 22
2.9.3 Calculation of the actions inside the wall 22
2.9.4 Calculation of the maximum forces acting on the wall 23
2.9.5 Sliding analysis 27
2.9.6 Overturning analysis 27
2.9.7 Load-bearing analysis 27
2.9.8 Calculation of ultimate pressure 27
2.10 SLIDING CHECK FOR SOIL REINFORCED STRUCTURES OR BLOCKS 28
2.11 INTERNAL STABILITY ANALYSIS (ANALYSIS OF THE SLIDING AND OVERTURNING BETWEEN A LAYER OF GABIONS AND A LAYER BELOW) 29
2.12 SETTLEMENT CALCULATION 29
2.12.1 Calculation of the loaded area 29
2.12.2 Calculation of the induced stresses 29
2.12.3 Calculation of the settlements 30
2.13 CALCULATION OF THE YIELD ACCELERATION 31
2.13.1 Application requirements 31
2.13.2 Yield Acceleration 31