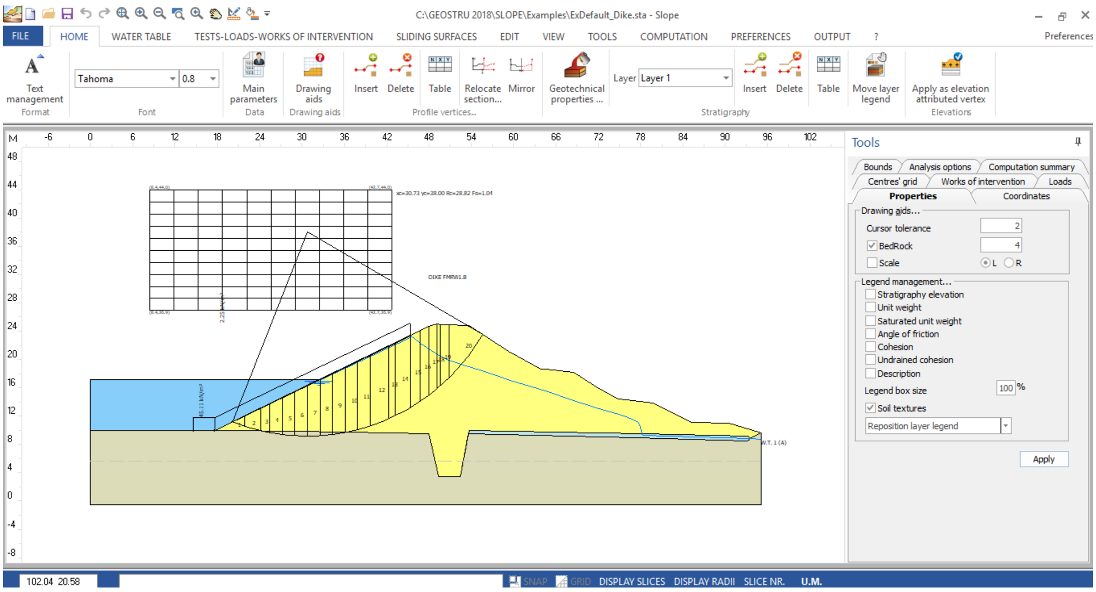
SLOPE

1 Slope

Slope stability software carries out the analysis of soil or rock slope stability both under static and seismic conditions, using the limit equilibrium methods of **Fellenius, Bishop, Janbu, Bell, Sarma, Spencer, Morgenstern & Price, Zeng Liang and (DEM) Discrete element's** method for circular and non-circular surfaces by which it is possible to ascertain slippages in the slope, examine a gradual failure, and employ various models of force-deformation relationship. Reinforcements with piles, gravity and/or reinforced concrete bracing walls, settings, geofabrics, anchors, and terracing may be specified. Distributed and point loads may be defined.



Slope stability

Supported computation standards

Verification analysis can be performed employing EN 1997-1, or classical approach (limit states, factor of safety) EN 1997 – option to choose partial factors based on National Annexes EN 1997 – option to choose all design approaches, consider design situations

DATA INPUT

• Graphic input with mouse
• Input from EXCEL files
• Input from DXF files
• Tabular numerical input
• Input of topographic profiles generated by TRISPACE

• Import of rasterized images
• Input ASCII files
• Input from Dynamic Probing

EMBANKMENT LOAD TYPES

• Point loads (inclined)
• Longitudinal loads

REINFORCEMENTS AND INTERVENTION WORKS

• Retaining walls
• Single piles, Sheet pile walls or Bulkheads
• Stabilization method: Broms limit load with automatic computation of the breaking moment of the section, Shear strength method, Zeng Liang's method
• Gabions
• Active and passive anchors
• Step terracing
• Reinforced earth: bars, strips and geotextile sheets
• Geogrids database that can be modified by the user
• Drained trenches
• Input of generic works
• Integrated template for the automatic generation of Wind turbines and telephone towers
• Nailed reinforcement implementation using Soil Nailing technique

COMPUTATION METHODS

• FELLENIUS (1936)
• BISHOP (1955)
• JANBU (1956)
• MORGENSTERN and PRICE (1965)
• SPENCER (1967)
• BELL (1968)
• SARMA (1973)
• D.E.M. (1992)
• ZENG LIANG (1995)
• Back Analysis
• Rock slope analysis using the Hoek and Brown method (1980)
• ANISOTROPIC ANALYSIS

NEUTRAL PRESSURES INCREMENT IN SEISMIC FIELD

In case of seism is estimated the increment of the neutral pressures produced by the deformations induced by the seismic waves. The formulas used are: Matsui et al., 1980, Seed & Booker, 1997, Matasovic,1993. All the parameters necessary for the estimate such as Arias index, Trifunac duration, etc. are calculated automatically by the program upon integration of the design accelerogram.

COMPUTATION OPTIONS

• Recalculate function to evaluate the safety factor of a specific surface with center X0,Y0 and radius R
• Identification of the critical slide surface, though automatic calculation
• Computation of the safety factor for surfaces which pass through two given points and are tangential to a straight line whose gradient varies automatically
• Automatic computation of the safety factor for surfaces that are tangential to a straight line vector
• Computation of the safety factor for surfaces which pass through either three, or one given point
• Differentiation between flexible and rigid retaining structures
• Stability analysis of submerged slopes (e.g. hillside lakes)
• Analysis of irregular surfaces
• Presence of seismicity and aquifers
• Stratified terrains and relative pore pressures

GRAFIC OPTIONS

• Display of the safety factor isolines
• Colored display of all sliding surfaces divided by a safety factor (to each color are assigned the safety factors in a fixed interval)
• Selection of the surfaces to be printed
• Options <Delete mesh>, <Move mesh> and numerical assignment of the centers' mesh
• Option <Translate groundwater> which allows to raise or lower the water table (very useful command for the sensitivity of the Fs when the groundwater level varies)
• Layer filling with textures or colors (the textures can be defined by the user)
• Graphical and numerical input for non-circular sliding surfaces
• Tools for inserting text, lines and polygons on the graphic sheet

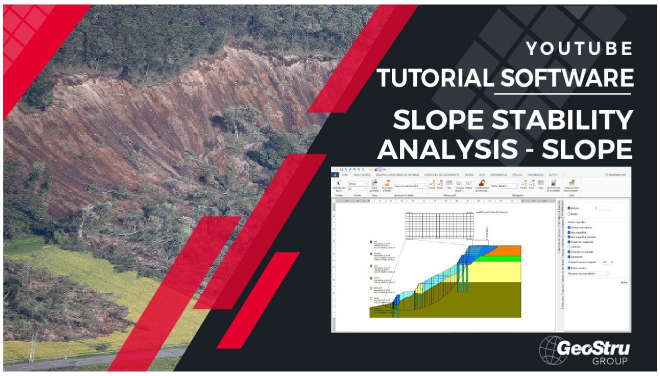
Additional features

Dynamic Analysis: Numerical method for the analysis of slope stability under seismic conditions for direct integration and modal superposition using Newmark's method (1965). It allows the computation of permanent displacements of the landslide mass by integrating the relative acceleration. It is also possible to generate the artificial accelerograms or import accelerograms from: SIMQKE and Sabetta F., Pugliese A.: Estimation of Response Spectra and Simulation of Non-stationary Earthquake Ground Motions.

Slope 3D: Generation of digital 3D models from GIS, DXF or Text files. Import files from SRTM (SRTM is a software created by GEOSTRU, included in the GEOAPP free suite, which allows the generation of a 3D model by simply selecting an area on Google Maps) of from GeoStru Maps, free app for Geostru users. The sections to be analyzed with Slope are automatically created in a dynamic way by moving on the 3D model.

DEM – Discrete Element Method: Advanced numerical method for the analysis of slope stability in static and dynamic conditions. Very sophisticated models of computation of linear and nonlinear analysis based on the behavior of the ductile or fragile soil.

M.R.E. *(Mechanically reinforced earth)*: Design and verification of reinforced soil retaining structures. Are carried out verification at: pullout and break for bar or strips reinforcements and geosynthetics, local stability (Tieback), global stability (Compound), sliding verifications of a rigid body, limit load, overturning. Provided standards: NTC2008, GRI (Geosynthetic Research Institute), BS8006/1995 (Code of practice for strengthened/reinforced soils and other fills), FHWA (Federal Highway Administration).



1.1 Important notes

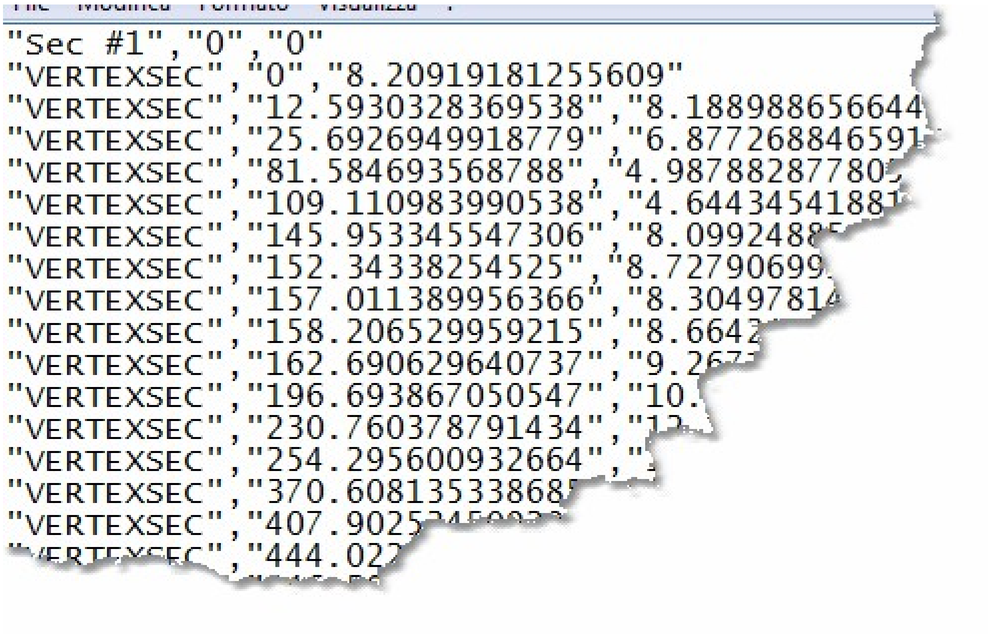
To use the software in a correct manner, it is necessary to respect some rules, listed below:

1. Reference system: the slope must be defined in the positive quadrant of a Cartesian reference system X, Y.
2. The Y dimension must be increased from left to right.
3. The sections inserted according to other reference systems than the one imposed may be mirrored using the command Mirror.
4. The distance between the minimum ordinate Y of the profile's vertices and the depth of the bedrock constitutes a constraint on the research of the safety factor (will not be taken into consideration the sliding surfaces that cut the aforementioned).
5. The geotechnical characteristics of the layers that make up the slope to be examined should be assigned starting from the upper layer to the lower one.

1.2 Data import

Import 3D model
Allows the import of text files that contain the three-dimensional information (x, y, z) of the points. The import system allows the import of text files in any format: it is enough to set the separator type (“,” or “;”), the first row to read and the numbers of the columns containing x, y and z. The command “*File data extraction*” applies a filter to the data contained in the text file and extracts the coordinates of the points on which to perform the triangulation. Through the “Triangulate” button it is generating the digital model on which bi-dimensional verification sections can be created. The sections are exported for Slope using the “*Assign the current section to Slope*” button.

Import TriSpace sections
TriSpace is the topography software of GeoStru that allows the creation of elevation plans, contoured elevation plans, 3D representation, 2D and 3D sections. The sections are exported from TriSpace in text files with *.sec extension, having the following format: VERTEXSEC, x, y. In the figure below you can see an example file generated from TriSpace.


"Sec #1","0","0"
"VERTEXSEC","0","8.20919181255609"
"VERTEXSEC","12.5930328369538","8.188988656644
"VERTEXSEC","25.6926949918779","6.877268846591
"VERTEXSEC","81.584693568788","4.98788287780
"VERTEXSEC","109.110983990538","4.6443454188
"VERTEXSEC","145.953345547306","8.099248
"VERTEXSEC","152.34338254525","8.72790699
"VERTEXSEC","157.011389956366","8.3049781
"VERTEXSEC","158.206529959215","8.664
"VERTEXSEC","162.690629640737","9.2
"VERTEXSEC","196.693867050547","10.
"VERTEXSEC","230.760378791434","
"VERTEXSEC","254.295600932664","
"VERTEXSEC","370.60813533868
"VERTEXSEC","407.9025
"VERTEXSEC","444.02

Example file generated from Trispace ready to be imported in Slope

Import DXF sections

The DXF must contain exclusively open polylines numbered from left to right, that define the topographic profile and the layers. Each polyline must belong to a specific LAYER.

Example: *Profile polyline on LAYER=0, Layer 1 on LAYER=1, Layer 2 on LAYER=2, Groundwater on LAYER=GROUNDWATER.*

Import sections from Penetrometric tests

The Dynamic and Static Probing software allows to connect the soil tests along a path and export them in an *.esp file. The esp file contains geometrical information (x, y, z) on the test and information on the test itself (number of blows, resistance, stratigraphy).

The topographical location is performed automatically, while the reconstruction of the stratigraphy is the responsibility of the operator, who will connect the layers belonging to the stratigraphic columns.

Import from LoadCap

LoadCap is a GeoStru software for the computation of bearing capacity and settlements of shallow foundations. For foundations on slopes it is necessary to perform the global stability analysis -the file exported from LoadCap contains all the information necessary to perform this analysis.

1.3 Data export

Export GFAS model

GFAS is a GeoStru software for the mechanical analysis of the soil using Finite Elements Method. The software allows the determination of the stress and strain state of each discretization element of the geotechnical model. The geometrical model used in Slope can be imported into GFAS for the analysis using Finite Elements.

Export works of intervention

the command allows the export of the geometry and stratigraphy of eventual support works inserted in the verification section. The exported support works are retaining walls: the exported file *.edc can be directly imported in the MDC software for the geotechnical and structural verifications of reinforced concrete and gravity walls. Together with the import the geotechnical characteristics and the stratigraphy of the terrain surrounding the support work is read as well.

1.4 Main parameters

Area

To find the area you can enter the address by separating the fields with commas.
For example: City, State. Or you can enter the coordinates in WGS84 system. To locate the site, you need to press the search button. The location of the site is an information that is included in the final report.

Soil type (Lithotype)

Soil slopes or Rock slopes.

⊞ Rock slopes

For rock slope, unlike for soil slopes, the Mohr-Coulomb failure criterion cannot be used to define the resistance of the material; however, with this method can be described a procedure that allows the application of classical Limit Equilibrium methods even for rocky slopes.

Surface form

The analysis can be conducted for circular surfaces as well as for generic shape/polygonal surfaces.

⊞ Surface form

For circular surfaces should be introduced the grid of the centers, while the polygonal/ free form surfaces must be assigned by points.

Acceptable level of safety

The data has no influence on the numerical calculation. Based on the value inserted to the software will highlight in various reports (text reports or graphic reports) the surfaces with a safety factor lower than the value set. So it is an indicator of the level of safety that the user wants to keep in reference to the limit state that is verified.

Safety factor search step

This data is important for the search of the safety factor when using circular surfaces. Having a fixed center, the search method of the critical surface is based on the analysis of possible surfaces with variable radius between a minimum and maximum value. The radius variation is done with an incremental step calculated as [(R_{max}-R_{min})/Safety factor search step].

BedRock depth

Depth of the rigid layer. The depth is estimated from the minimum ordinate of the profile (in presence of layers, minimum ordinate of the layers). The search for the critical surface takes place between the typographic profile and the BedRock.

Seismic action

In the pseudo-static analysis the earthquake is computed through the horizontal and vertical seismic coefficients, respectively k_h and k_v. According to the selected standard is possible to identify the seismic coefficients of the area.

> The seismic acceleration is required for the calculation of pore pressure in seismic field.

Increment of pore pressure

Selecting this option allows you to evaluate the pore pressure generated in the ground in the presence of water table and in conjunction with the occurrence of earthquakes. For the calculation of pore pressure is required to import an accelerogram on which the program automatically calculates the intensity of Arias, the

intensity of the intersections with the time axis and the period of Trifunac and Brady (1975). To import the accelerogram click the triangle next to "Accelerogram duration Trifunac": a dialog box opens allowing you to select the file (*.txt, *.cvs) in which is shown the values of the acceleration in *[m/s²]* and of time in *[s]*. Here, you can choose the conversion factor of time *t* and of acceleration *a* for the automatic conversion of values in the units required by the program.
Following importation, in Parameters, are calculated automatically the values required ? from processing.
It should be stressed that for the purposes of evaluation of the pore pressure, the user is required to insert additional geotechnical characterization of soils involved in this phenomenon: in the definition of the stratigraphy for each soil type, should be inserted the Additional data.

Partial factors geotechnical parameters
The partial factors that are introduced by the user are factors that reduce the geotechnical characteristics of the soils defined in the stratigraphy.
These coefficients generally apply to the "characteristic" parameters that the user enters in the stratigraphic modeling of soil.
The calculation of the safety factor on the identified surfaces is performed with the reduced parameters of soil strength only if is selected the option "Use these coefficients to reduce the strength of the material".

Partial factor on the resistance
The coefficient reduces the resistance mobilized along the potential sliding surface. The value of the coefficient influences numerically the computation of the safety factor defined by the ratio between the limit strength available and that calculated at the base of each slice. Values greater than unity reduce the available strength of the soil by decreasing the factor of safety.
In the stability analysis, it is advisable to insert a "Partial resistance factor" equal to 1,1 and assign a unit value to the "Acceptable level of safety". With the above assumptions, the user retains a margin of safety on all surfaces that return by a safety factor greater than or equal to unity.

1.5 Drawing aids

Drawing aids button allows you to customize the grid type in the working area and the related snap.

Attention:

The tolerance of the cursor is very important as it represents the sensitivity of the mouse around the graphical objects, whether they are support works or vertices of points.
To move and/or modify an object or a vertex (profile, layers, groundwater, loads, etc.) position the mouse pointer near the object to be modified; When the pointer changes form you can make the changes by clicking on the object. You can modify the element only if the mouse pointer is contained within the radius defined by Cursor tolerance.

The tolerance is assigned in the base of the profile dimensions. For example, for dimensions of the order of 100 meters assign a tolerance between 0.5-1.

1.6 Text management

Text management command allows the customization of font and dimensions of texts. The **Default** button makes available, for different work files, the style chosen by the user. The first button on the left aligns the texts to the first style available in the list (Layer legend).

Free texts are the ones entered by the user that do not belong to previous categories (Layer legend, Elevation/distance table, etc.). They are also used for the representation of the number of vertices in the phase of graphical input.

1.7 Penetration tests

This command allows you to import static and dynamic penetration tests processed with Dynamic and Static Probing, displaying, in the first case, the diagram of the number of blows and the stratigraphic column, and the trend of the tip resistance and the stratigraphy, in the second case.
After selecting the command just click on the insertion point and displayeds the window where you can select the file to insert (in .edp format - format of the Static and Dynamic Probing files). The chart can be moved with the mouse while keeping the button pressed after performing a single click on it.

1.8 Inserting vertices

The commands below are available for the vertices of polylines of: topographic profile, layers, groundwater, water table.

Inserting and editing

To insert a vertex graphically select the Insert command, move to the work area and click the left mouse button.

The position can be modified numerically in Vertices table displayed to the right of the workspace.

After inserting all the vertices of the polyline confirm the entry with the right click.

To change the position of a vertex select the Insert command, move the mouse on the point and drag it to a new location.

Delete

To delete a vertex, select the Delete command, move the mouse on the vertex to be deleted and click the left mouse button.

For proper operation of: insert, edit and delete correctly set tolerance of the cursor in drawing aids.

To delete multiple vertices simultaneously: select the delete command, press the left mouse button and with the button pressed, move to a new location, a rectangle will be drawn. All vertices within the rectangle will be deleted.

Table

The vertices can be assigned numerically using the Table command. In numeric input right-click on the input grid to import, copy and export the data.

You can paste the vertices in the Table, having them previously copied from a text editor. You can copy individual values or entire sequences - in this case separate fields with a tab.

1.9 Output

Create a report

Create reports, at the same time you can choose what to include in the report using the command *Report print options*.

Export in Dxf

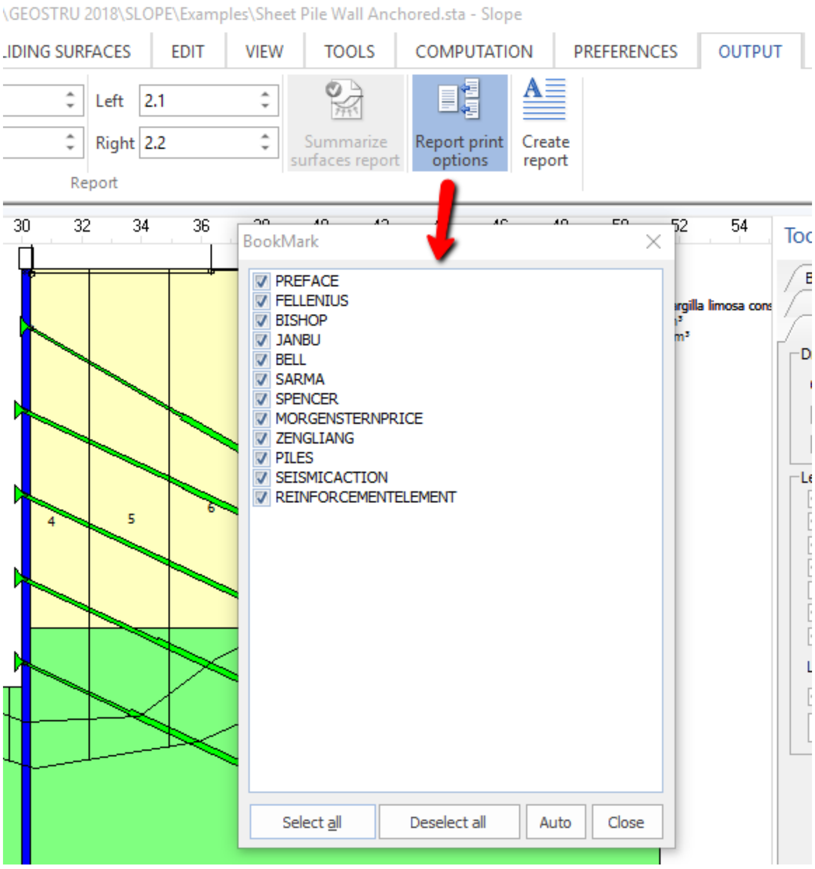
Realizes the drawing of the work area (foundation, layers, legends, etc.).

Export Bitmap

Creates an image of the work area.

Report print options

Pressing "*Report print options*" command shows the BookMark window from which the user can choose the topic to insert in the theoretical report by flagging the related checkbox.
After performing the stability analysis of a slope, by pressing the "*Auto*" button the software will make an automatic search of the theoretical references to introduce in the report on the base of the computation options chosen by the user.



1.10 Geotechnical properties

This button opens a window that brings together all the data related to the geotechnical characterization of slope. The geotechnical parameters to be entered must be assigned beginning from the upper layers (see also § Conventions).

Nr.: Number of the layer 1, 2, 3, 4, etc.

DB: Soils database with associated geotechnical characteristics.

Unit weight: Unit weight of the layer in a specified unit of measurement; in case of a submerged layer, please insert Saturated unit weight.

Saturated unit weight: Saturated unit weight of the layer in specified unit of measurement.

Cohesion: Cohesion of the soil in the specified unit of measurement. In the presence of water table, for the analysis in undrained conditions, the value for the Undrained cohesion must be inserted.

Peak friction angle: Represents the angle of soil strength in degrees; in the presence of water table enter the effective parameter. For undrained analysis insert zero.

Residual friction angle: Is the angle of resistance of the soil in degrees when it is already mobilized the landslide; this parameter is necessary in the DEM method for analysis with the redistribution of the stresses.

K modulus (Normal and tangential stiffness) : Winkler modulus of the soil in the specified unit of measurement, parameter required for analysis by the DEM method (Discrete Element Method).

Permeability: Specify whether the layer is permeable or impermeable; In the presence of the confined ground water table it must define permeable the layer in which is located the aquifer and assign the relative piezometric.

Textures: Move to this cell and click with the right mouse button: will be displayed the color palette to choose from and associate with the corresponding layer. Alternatively, you can assign the textures on the right side of the dialog box: choose with a click of the mouse and the while pressing the mouse button, drag it in the cell relative to the layer.

The textures displayed to the right side of the "Geotechnical properties" window are installed separately through the file Texture provided by GeoStru. They are external to the program and can be modified or integrated acting on their installation folder. You can also change them directly from the program by opening an internal editor with a double click on the texture to modify.

Attention:

If the texture list is empty, install the texture file, or set the correct paths from Preferences.

Description: Move to the cell and write a text; it will appear in the legend of the layers.

Geotechnical parameters to use. Angle of friction

- *peak friction angle:* this parameter is recommended for sand and gravel with a high degree of densification (relative density> 70%) or in any case in slopes where the landslide is not mobilized;
- *residual friction angle*: this parameter is recommended for checking slopes in a landslide;
- *critical state friction angle*: this parameter can be estimated from the peak friction angle through a relation proposed by Terzaghi and it is advisable for slightly thickened sands and gravels (relative density <20%).

Anisotropic strength

Additional data: shear modulus, density, plasticity index, etc. are necessary to calculate the pore pressure increment seismic field.

1.10.1 Additional data

Additional data: Shear modulus, density, plasticity index, etc. are necessary to calculate the increase of pore pressure in seismic field.

Geotechnical behavior: noncohesive, cohesive, noncohesive - cohesive

Dynamic low strain shear modulus: represents the shear modulus at low levels of strain. The threshold is generally set between 0.0001% and 0.001%.

Dynamic shear modulus: represents the shear modulus beyond the threshold of linearity, where the soil has a highly non-linear and dissipative behavior, with a reduction of shear stiffness G.

Relative density: for granular soils this parameter expresses the degree of densification between the particles. It depends on the uniformity or the variability of the particle diameters: more variable the diameter of the soil, the higher will be the relative density. A soil classification based on relative density is shown in the table below.

Relative density *(%)*	Description
0 - 15	Very low density
15 - 35	Scarcely densities
35 – 65	Avrage densities
65 - 85	densities
85 - 100	Very densities

Classification of soils according to their state of densification

***Over-consolidation ratio*:** is expressed by the parameter OCR and represents the relation between the preconsolidation pressure and the geostatic pressure. Its value is greater than unity for overconsolidated soils.

***Plasticity index*:** is a parameter of the behavior of cohesive soils. Its value is given by the difference between the liquid limit and plastic limit (Atterberg limits).

***Number of load cycles required to produce liquefaction*:** represent the number of cycles of loading and unloading which trigger the liquefaction, ie cancel the effective stress state of the soil (will be calculated automatically by the program).

Theoretical notes Pore pressure

1.10.2 Anisotropic

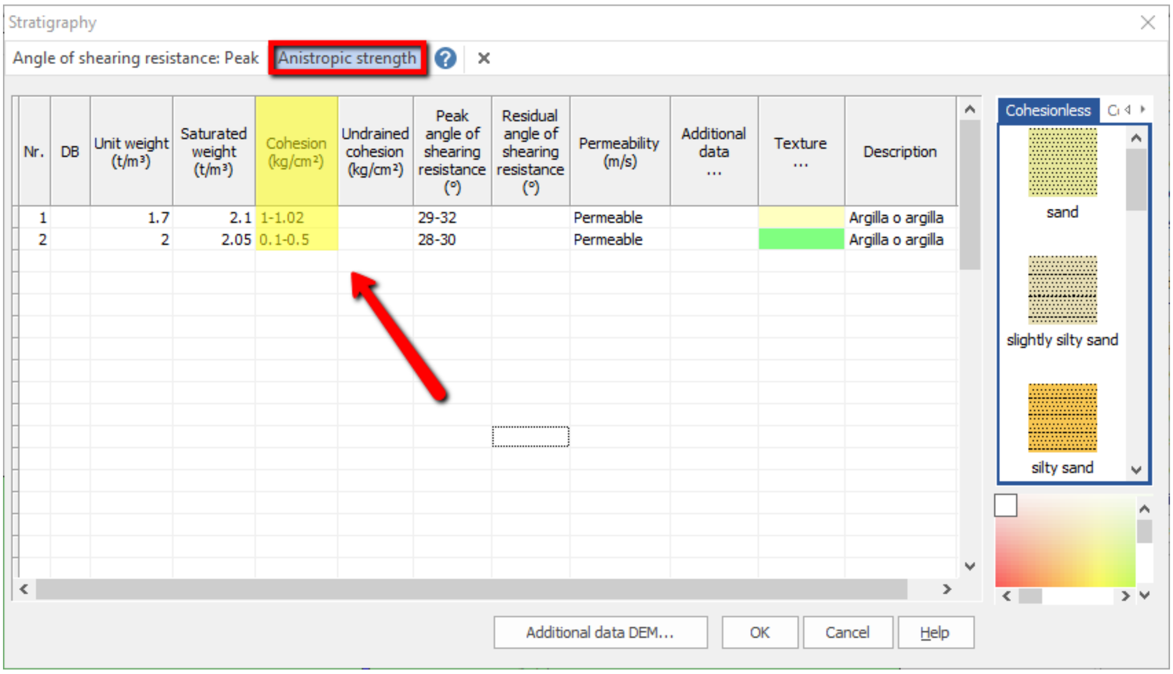
This strength model uses the following equation for dealing with anisotropy in the soil strength:

$$c=c_h(\cos\alpha_i)^2+c_v(\sin\alpha_i)^2$$

$$\varphi=\varphi_h(\cos\alpha_i)^2+\varphi_v(\sin\alpha_i)^2$$

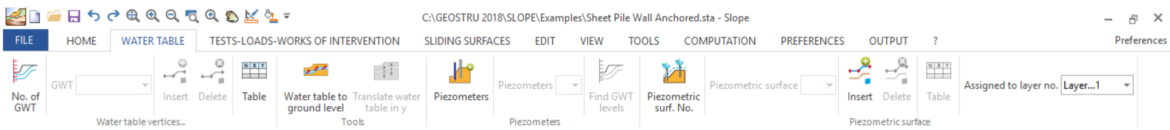
The subscripts stand for horizontal and vertical. The horizontal and vertical components are specified. Alpha(α) is the inclination of the slice base.

For the anisotropic analysis, insert two values (horizontal , vertical) of the parameter geotechnical separated by - ex: c_h-c_v for cohesion (100-120)



1.11 Groundwater table

The commands for managing the GWT are represented in the image below:

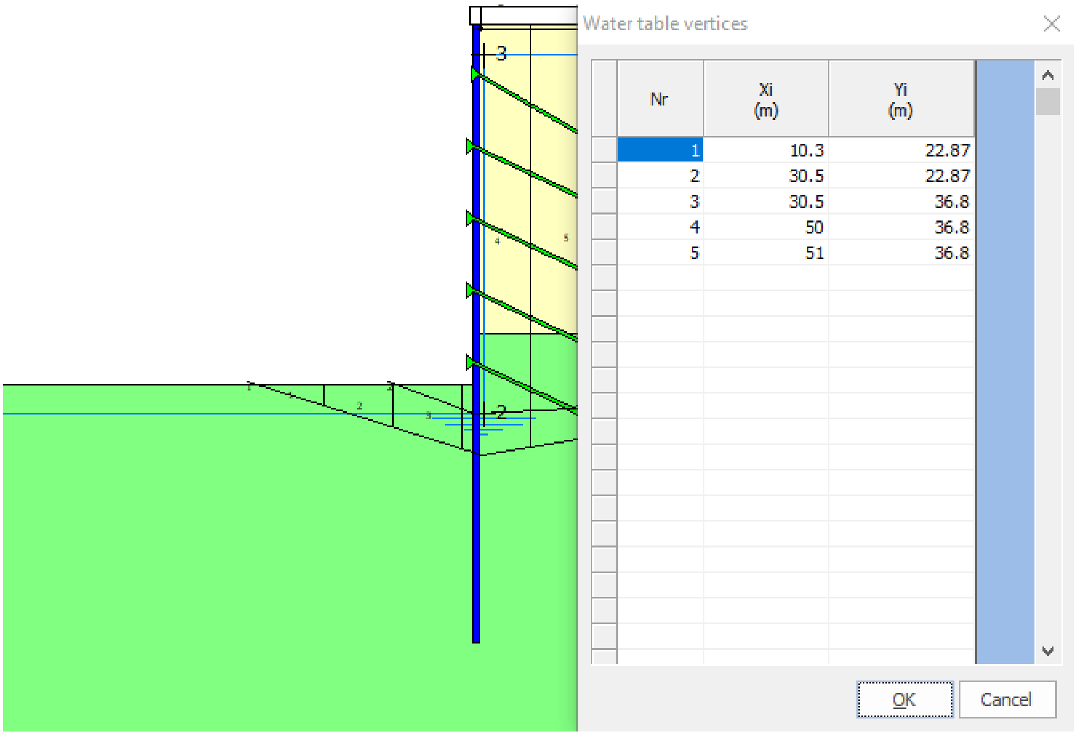


Nr GWT: This command gives the possibility to create (typing a number greater than one) the number of GWT corresponding to the number entered, or delete (by typing zero) all GWT previously created. Entering the number of GWT to create is activated the GWT window, where each GWT is indicated with an integer number.

Insert (GWT vertices): This command allows the definition of GWT by inserting the points with the mouse directly on the profile.

Delete (GWT vertices): This command activates all vertices - just click on the ones you want to delete. Right click to apply the modifications.

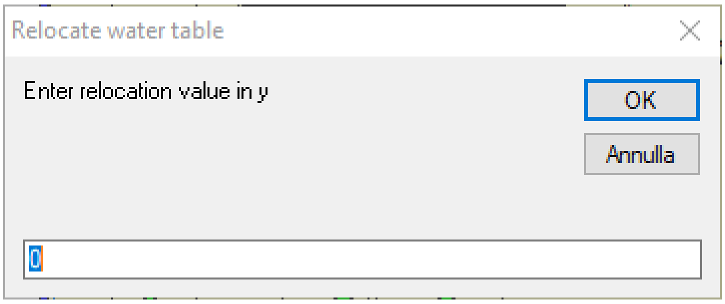
(GWT vertices) Table: This command opens a table where the vertices of the GWT can be managed. In the first column the vertices of the GWT are represented with integer numbers from 1 to n, the second and the third column contain the coordinates of the vertices with respect to the global reference system. If more GWT must be managed, a table is associated to each of them.



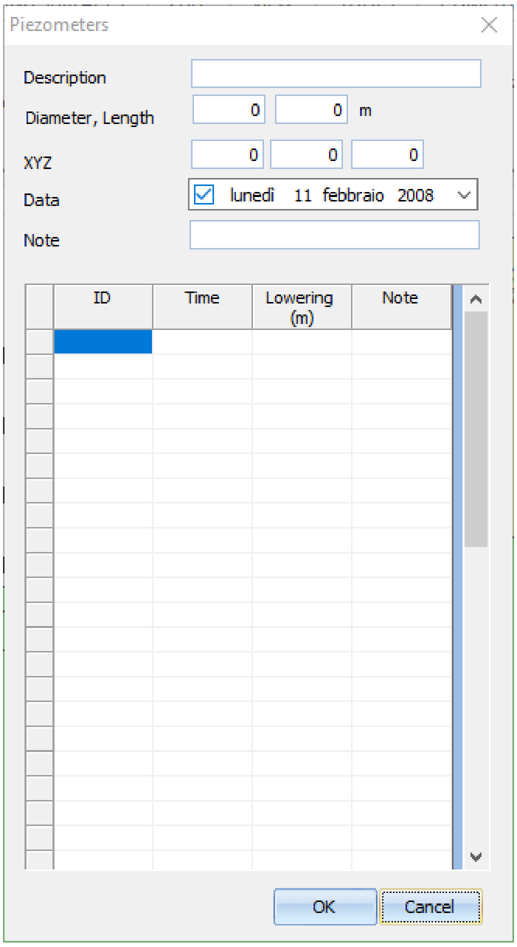
To define the GWT at least three points are needed.

GWT at ground level: Translates the GWT at ground level, and the piezometric will coincide with the soil profile.

Translate GWT in y: Translates the GWT along the y axis by inserting a number (bigger or smaller that zero) in the window below. For a value grater than zero the GWT is translated in upwards direction (towards the positive part of the axis), while for a value smaller than zero, GWT is translated in downwards direction (towards the negative part of the axis).



Piezometers: Often used to monitor the level of the GWT (responsible for pore pressures that act along the failure surface of the soil), the command allows the management of n piezometers. By assigning zero to this command, all piezometers previously defined will be deleted.
After assigning the number of piezometers to be used a window with a sequence of integer numbers is opened (numbers from 1 to n) to each number is associated a piezometer and to each piezometer is associated a table like the one in the image below.



The table will contain a range of information, including:

Description

: the field can be used to describe the characteristics of the piezometer. Diameter, Length: These refer to the geometric dimensions of the mea suring instrument.

XYZ: These represent the coordinates of the insertion point (piezometer head) with respect to the global reference

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(reading order is from top to bottom). Each piezometer corresponds to a table with its own characteristics.

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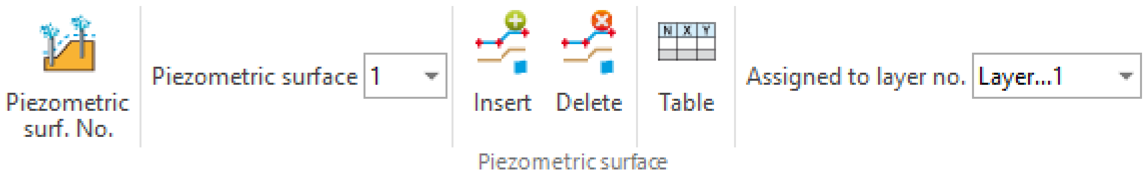
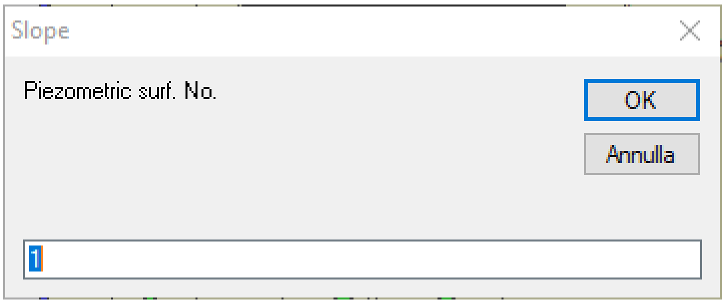
Find GWT levels: After inserting the piezometers this command allows interpolating the measuring points and draw a polyline that defines the level of groundwater.

> In order for the **Find GWT levels** command to work properly it is necessary that each piezometer inserted has the same number of readings.

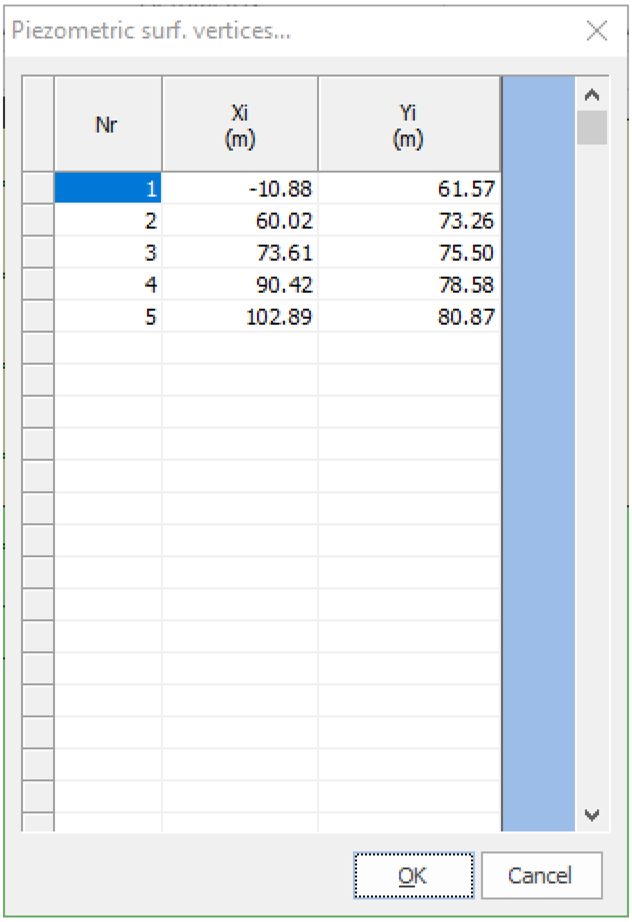
Slope allows the management of water under pressure through the command **Piezometric surf**.



Piezometric No: From this command can be created the piezometrics (inserting a number greater than one) or they can be deleted (by typing zero all piezometrics previously created are deleted). Entering the number of piezometers that must be managed, two windows are activated: **Piezometric /Assigned to layer no** from where the piezometric can be associated to a layer.



Insert (Piezometric vertices): This command allows the definition of the piezometric level by inserting the points with the mouse directly on the profile. While the vertices are inserted with the mouse a table for the vertices coordinates is filled in the right-side menu.



Nr	Xi (m)	Yi (m)
1	-10.88	61.57
2	60.02	73.26
3	73.61	75.50
4	90.42	78.58
5	102.89	80.87

Delete (Piezometric vertices): This command activates all vertices of the chosen piezometric level - just click on the ones you want to delete. Right click to apply the modifications.

1.12 Elevations

The tool allows you to set the elevations for any element: vertices of profile, layers, groundwater table. To insert the elevations selects the command and click on the vertex you need the elevations for. If the vertex has an elevation, at the next click the elevation is canceled. Block commands can be performed by using the popup window from the right bottom side.

1.13 Loads

To insert the loads on the embankment proceeds as follows:

1. Select **Insert** button.
2. Move the mouse on the work area and press the left mouse button on the point of insertion.
3. In the Loads panel:

- modify if necessary the coordinates X_i, Y_i which identify the insertion point;
- the load value in the specified unit of measurement in F_x, F_y and press Apply.

Modify load: Move the mouse to the insertion point of the load - in the Load panel will be displayed the characteristics of the load change them and press Apply.

Delete load: Move the mouse to the insertion point of the load; when the cursor changes form, press the left mouse button, the load will be deleted.

Loads scale: Allows you to define a scale for displaying loads.

1.14 Works of intervention

Retaining walls

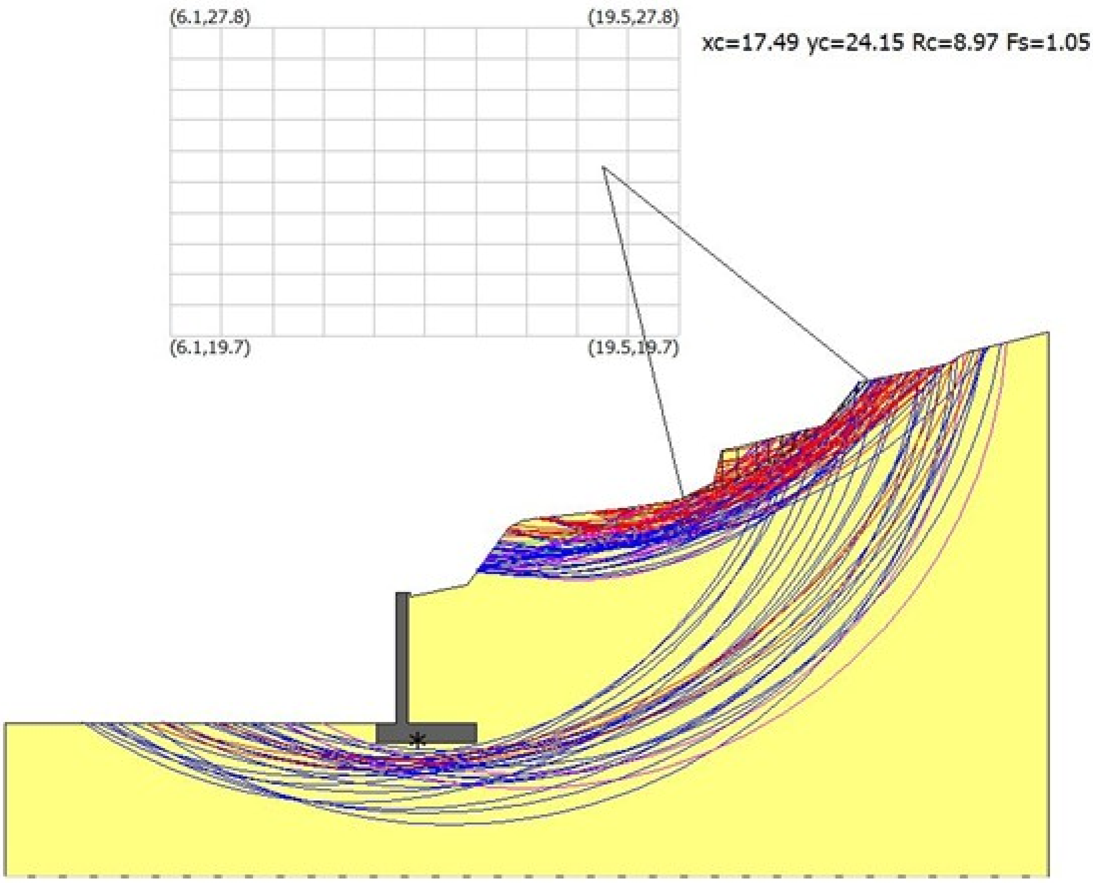
To insert retaining walls on the slope proceeds as follows:

1. Define the types in Walls in the **Intervention works** definition panel, on the right side of the screen.

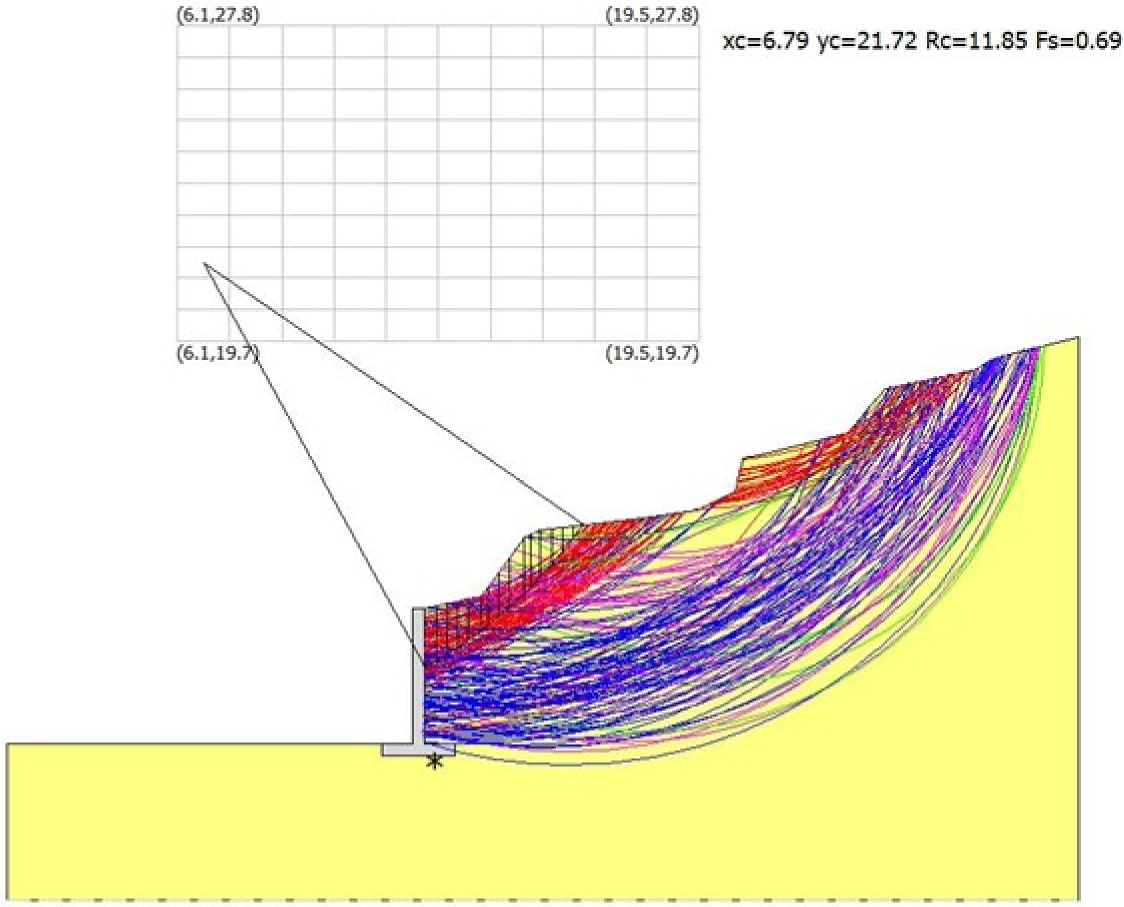
2. After defining one or more walls, in the same panel you will see active the icon related to the insertion of the wall (a wall with the + sign on the left side).
3. Go with the mouse in the work area and left click on the insertion point.
4. To insert the coordinates you can use the panel on the right side of the screen. Choose **Modify** (second wall icon) and insert the new coordinates in the table (or simply move the wall in the work area).
5. Click **Apply** button to confirm.

To **delete** a wall, choose the last of the three wall icons (wall with x sign on the left side) and click on the wall to delete.

It is possible to exclude the sliding surfaces that intercept the wall. In order to apply this condition, it is necessary to access the intervention panel and the window in which the wall geometry is characterised, choose the **non-deformable type and do not select the automatic profile modification option**. The result obtained is shown in the figure below.



If the **automatic profile modification option is selected**, the programme also considers sliding surfaces that interfere with the wall.



Anchors

To insert anchors proceeds as follows:

1. Define the types in Anchors in the **Intervention works** definition panel, on the right side of the screen.
2. After defining one or more anchors, in the same panel you will see active the icon related to the insertion of the anchor (an anchor with the + sign on the left side).
3. Go with the mouse in the work area and left click in the insertion point.
4. To insert the coordinates, you can use the panel on the right side of the screen. Choose **Modify** (second anchor icon) and insert the new coordinates in the table (or simply move the anchor to the work area).
5. Click **Apply** button to confirm.

To **change their position** choose the Edit command (central icon) from the Intervention Works panel and position yourself on the wall to be moved, assign the new position in the Work Status frame displayed at the bottom of the panel.

To **delete** an anchor, choose the last of the three anchor icons (anchor with x sign on the left side) and click on the anchor to delete.

Pilings

To insert piles proceeds as follows:

1. Define the types in Pilings in the **Intervention works** definition panel, on the right side of the screen.
2. After defining one or more piles, in the same panel you will see active the icon related to the insertion of the piles (a pile with the + sign on the left side).
3. Go with the mouse in the work area and left click on the insertion point.
4. To insert the coordinates you can use the panel on the right side of the screen. Choose **Modify** (second pile icon) and insert the new coordinates in the table (or simply move the pile to the work area).
5. Click **Apply** button to confirm.

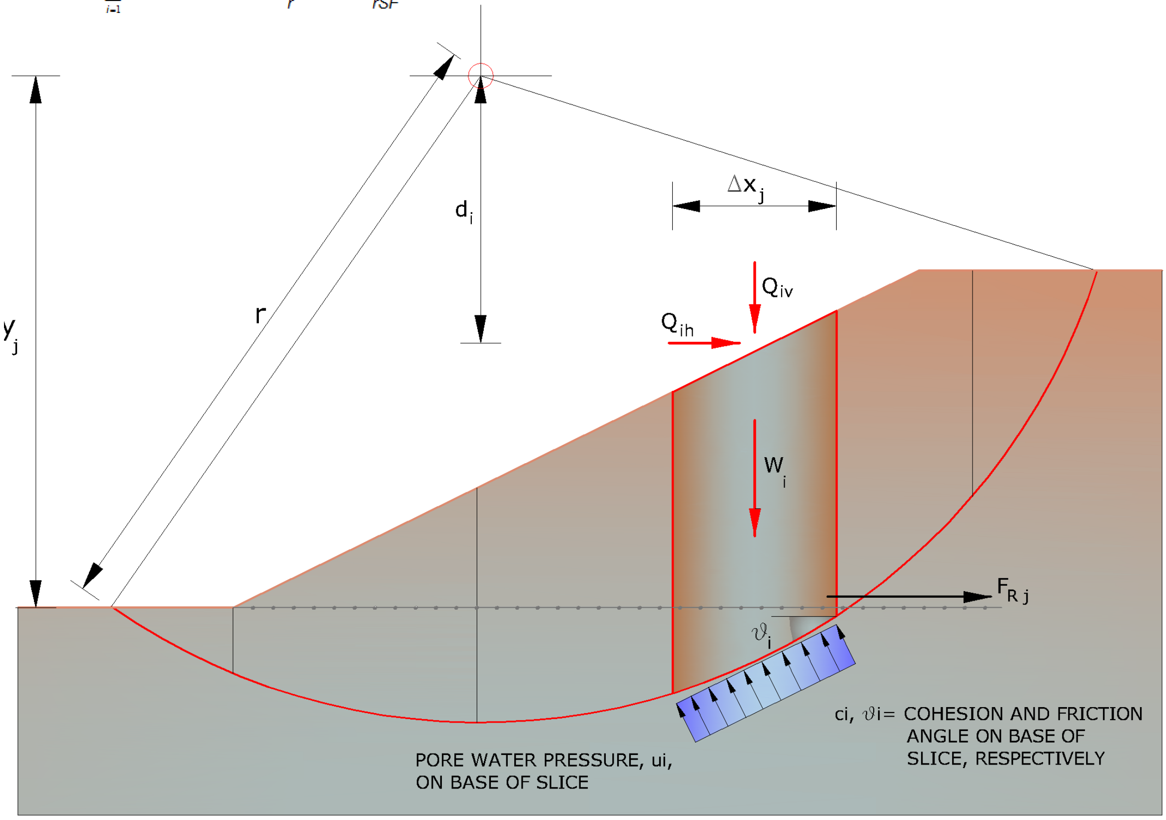
To **change their position** choose the Edit command (central icon) from the Intervention Works panel and position yourself on the wall to be moved, assign the new position in the Work Status frame displayed at the bottom of the panel.

To **delete** a pile, choose the last of the three pile icons (pile with x sign on the left side) and click on the pile to delete.

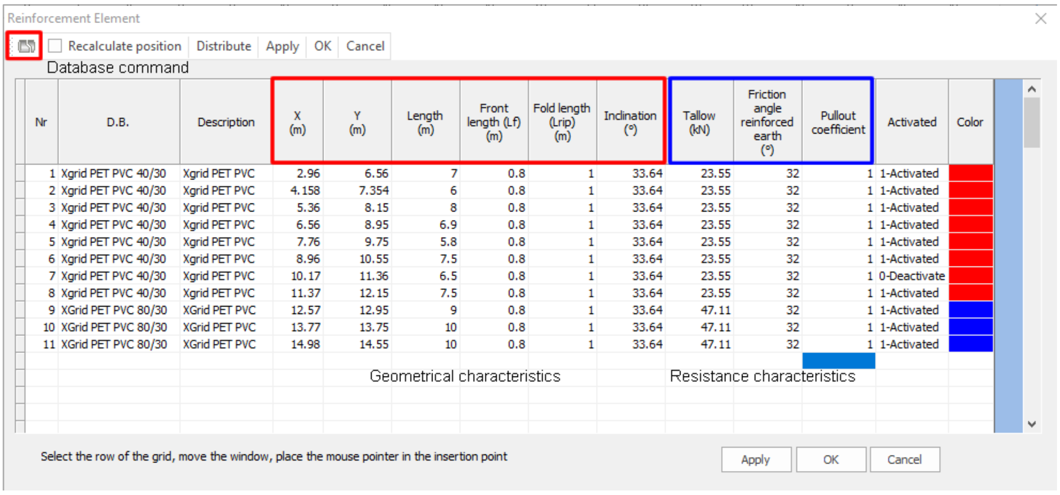
1.14.1 Reinforcing element

The reinforcing element (generally a geotextile) is placed on the ground to confer a greater resistance to sliding along a probable failure surface, see image below.

$$SF = \frac{\sum_{i=1}^{n} \left[\frac{c_i \Delta x_i + (W_i + Q_{iv} - u_i \Delta x_i j \tan\phi_i)}{\cos\theta_i + (\sin\theta_i \tan\phi_i)/SF} \right]}{\sum_{i=1}^{n} (W_i + Q_{iv}) \sin\theta_i - \frac{\left(\sum_{i=1}^{n} Q_{ih} d_i j\right)}{r} - \frac{\left(\sum_{j=1}^{m} F_{Rj} y_j\right)}{rSF}}$$



Clicking on the command ▸"Reinforcing element"



Reinforcement Element

Recalculate position | Distribute | Apply | OK | Cancel

Database command

Nr	D.B.	Description	X (m)	Y (m)	Length (m)	Front length (Lf) (m)	Fold length (Lrip) (m)	Inclination (°)	Tallow (kN)	Friction angle reinforced earth (°)	Pullout coefficient	Activated	Color
1	Xgrid PET PVC 40/30	Xgrid PET PVC	2.96	6.56	7	0.8	1	33.64	23.55	32	1	1-Activated	
2	Xgrid PET PVC 40/30	Xgrid PET PVC	4.158	7.354	6	0.8	1	33.64	23.55	32	1	1-Activated	
3	Xgrid PET PVC 40/30	Xgrid PET PVC	5.36	8.15	8	0.8	1	33.64	23.55	32	1	1-Activated	
4	Xgrid PET PVC 40/30	Xgrid PET PVC	6.56	8.95	6.9	0.8	1	33.64	23.55	32	1	1-Activated	
5	Xgrid PET PVC 40/30	Xgrid PET PVC	7.76	9.75	5.8	0.8	1	33.64	23.55	32	1	1-Activated	
6	Xgrid PET PVC 40/30	Xgrid PET PVC	8.96	10.55	7.5	0.8	1	33.64	23.55	32	1	1-Activated	
7	Xgrid PET PVC 40/30	Xgrid PET PVC	10.17	11.36	6.5	0.8	1	33.64	23.55	32	1	0-Deactivate	
8	Xgrid PET PVC 40/30	Xgrid PET PVC	11.37	12.15	7.5	0.8	1	33.64	23.55	32	1	1-Activated	
9	XGrid PET PVC 80/30	XGrid PET PVC	12.57	12.95	9	0.8	1	33.64	47.11	32	1	1-Activated	
10	XGrid PET PVC 80/30	XGrid PET PVC	13.77	13.75	10	0.8	1	33.64	47.11	32	1	1-Activated	
11	XGrid PET PVC 80/30	XGrid PET PVC	14.98	14.55	10	0.8	1	33.64	47.11	32	1	1-Activated	

Geometrical characteristics

Resistance characteristics

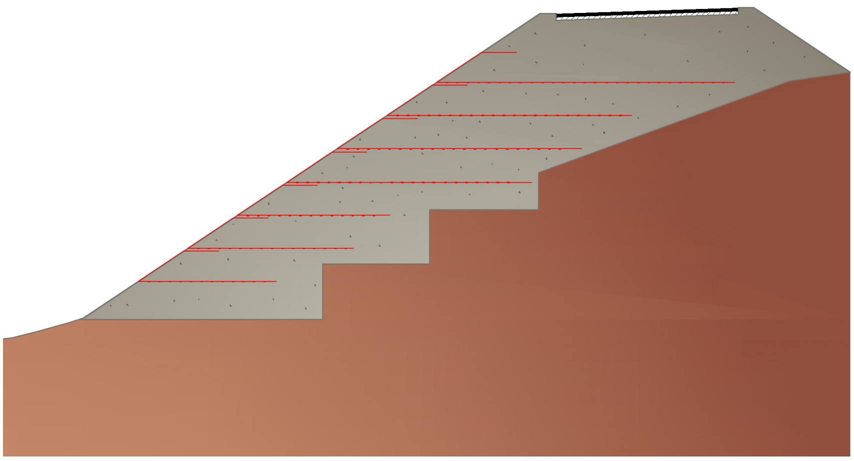
Select the row of the grid, move the window, place the mouse pointer in the insertion point

Apply OK Cancel

Appears a table in which one must define the geometric characteristics and resistance of the element.

When the reinforcement element has not been attached to the profile of the soil but moved along its horizontal axis, inserting the flag to **"Recalculate position"** and pressing first "Apply" and then "OK" the software automatically repositions the element properly and then can be associated with a color and make it active or not depending on the computational requirements.
Generally, for stabilizing a road embankment, see fig., is used the technique of reinforced earth; in order to model it you can proceed as follows:

- clicking on **"Distribute"** opens a dialog box asking how many reinforcements to be distributed on the slope profile
- typing the numerical value, the software will place in all the reinforcements of known geometric features
- the user may at any time modify any characteristics of the reinforcements - to confirm the modification, simply click on the button "Apply" and then "Ok".



1.14.2 Walls types

At this point, it is possible to define different types of retaining walls to be inserted on the profile of the slope. To define a new type, press the New button and assign the geometrical data and the specific weight required.
To edit an existing type, just select it by sliding the various types with the Next button and make the desired changes. When inserting the data, it is available the option to consider or not the flexibility of the work (*Deformable* or *Non-deformable*); are considered rigid RCC works, while works such as gabions or walls in stone are considered flexible. For flexible works, it must be assigned the shear stress of the work

in the specified unit of measurement, in this case in the calculation of *FS* will be considered also the sliding surfaces which intersect the work and taken into account the resistance opposed by it to sliding. If the support work inserted is rigid, the same surfaces (the ones that intersect the work) are automatically excluded from the calculation and are considered as stabilizing effect only the weight of the work.

⚠ **Note:**

By inserting a wall on a slope the software automatically changes the profile of the slope adapts it to the geometry of the wall; if the user does not want to automatically modification of the profile must disable the option shown at the bottom left of the dialog box "Automatic profile modification".

1.14.3 Pilings

The selection of the command displays a dialog box in which the following data is required:

Nr: Serial number of the piling.
Description: Identification text of the pile chosen by the user.
Length: Insert the length of the pile.
Diameter: Insert the diameter of the pile.
Interaxis: Insert the transverse interaxis between the piles.
Inclination: Insert the inclination angle of the axis of the pile from the horizontal.
Share strength: Insert the value of the shear strength of the pile section; this parameter is only taken into account if you choose as a stabilization method the shear stress (see next point).

Stabilization method: Choose between the two options proposed the way in which the pile intervenes on the stability of the slope: the method of the shear stress, with which the pile, if intercepted, opposes a resistance equal to the shear strength of the section, or the limit load method which considers as resistant strain the horizontal limit load relative to the interaction between the piles and the lateral ground in movement, function of the diameter and of the distance between the piles. For the considerations in the evaluation of the reaction of the soil using the method of Broms please refer to the bibliography.

For the limit load method must be inserted the yield moment (My) of the section

⊞ Automatic computation My

The program will automatically calculate My (yield moment) , for default pile diameters and reinforcements that can be selected from the drop-down menu displayed after pressing Yield moment button. The value of the yield moment is necessary when choosing the method of limit load as a mechanism of resistance of the pile on the stability of the slope. On the limit load is possible to assign a safety factor that will be applied by the software on the calculation of the resistance opposed by the pile to sliding.

1.14.4 Anchors

Anchor models, like other support works, must be defined before being used. The relative dialog window presents a table in which the different anchor models can be defined using the following characteristics:

No: Sequence number of the typology
Description: work description
No. of series/Step: the typology can be made up of one or more anchors: in the firs case insert 1; in the case of a series of anchors or nails it is possible to insert the number and the step, separated by the "/" character. In this last case the program generates a series of n anchors with the same characteristics.

Example*: 10/0.5 is equivalent to a series of 10 anchors with a 0.5m spacing step between them.*

The other items are necessary to define the geometry of the structural element.

The different typologies are: Active anchor, Passive anchor, Nailing. For each of these the work ultimate resistance must be assigned and will condition the stabilization according to the following cases:

Case 1

The sliding surface does not intercept the anchor (neither the free length, nor the foundation): in such a case is considered no contribution of resistance;

Case 2

The anchor is intercepted in the free length, so the foundation is anchored to the stable part: the tensile stress is considered as 100% resistant action and is inserted on the base of the slice that intercepts a force equal to the tensile stress. This force is subsequently decomposed into normal and tangential components, and the tangential component is inserted as a contribution to the shear strength on the sliding surface.

Case 3

The anchor is intercepted on the foundation, so the foundation comes into operation only for the resistant length over the sliding surface: the tensile stress, in this case, is considered with a defined percent of the ration between the resistant length and the length of the foundation. The action is treated like above.

For the cases 2 and 3 the traction refers to a section of unitary depth (dimension orthogonal to the section of the slope) as a function of the longitudinal distance (is multiplied by the distance).

Important notes:

Even if a series is intended, it is always assigned the ultimate strength of a single anchor or nail.

Note on works:

For active works, the resistive component of the work along the sliding plane is subtracted from the Driving Forces. For passive works, the resistive component of the work along the sliding plane is summed to the Resisting Forces.

Consolidation using Soil Nailing technique

The reinforcement technique of the soils using nails named “soil-nailing” consists of introducing reinforcements in the soil mass, having the function of absorbing stresses that the soil alone couldn’t be able to support.

The reinforcement system is a passive one; the soil adjacent to the reinforcement, at the time of its installation, is practically not solicited.

Resistance: the pullout resistance of the nails developed on the mortar-soil interface can be calculated using Bustamante method.

1.14.4.1 Soil Nailing

Design method of Soil Nailing system

One of the interventions to stabilize a slope is that of soil nailing.

The sizing of the steel bars (internal verification) is performed assuming attempt dimensions for them and verifying that:

- The bars do not break due to tensile stress as a result of the imposed tensile stress;
- The bars do not slip off the mortar due to insufficient adhesion;
- The soil surrounding the bar does not break due to insufficient adhesion.

The safety factor (FS) is defined as:

SF = Available force / Force required

To estimate the maximum values ðf resistance can be used the relations proposed in the literature by *Hausmann 1992) and MGSL Ltd (2006).*

Maximum allowable tensile strength of the steel bar:

$$Ta = (\Phi \cdot f_y) \cdot (d - 4)^2 \cdot \pi / 4 \qquad \text{Eq (5.8)}$$

where

Φ	= reduction factor of the stress established by the legislation
fy	= steel yield strength
d	= steel bar diameter

Maximum allowable force between steel and mortar:

$$[\ \beta\ (f_{cu})^{1/2}] \cdot \pi \cdot (d - 4) \cdot Le / SF \qquad \text{Eq (5.9)}$$

where

β =0.5 for bars type 2 according to Australian standard (imposed by standard)

f_{cu} = compressive strength of concrete at 7 days

SF = adopted safety factor (imposed by standard)

Le = effective length of anchor

Maximum allowable force between the ground and mortar:

$[(\pi DC' + 2D\ K_{\alpha}\ \sigma_{v}'\ \tan\Phi)\cdot Le] / SF$ Eq (5.10)

where

D = diameter of the hole in the ground

C′ = effective cohesion of soil

K_{α} = coefficient of lateral pressure (α = angle of inclination) = $1 - (\alpha/90)(1-Ko) = 1 - (\alpha/90)(\sin\Phi)$

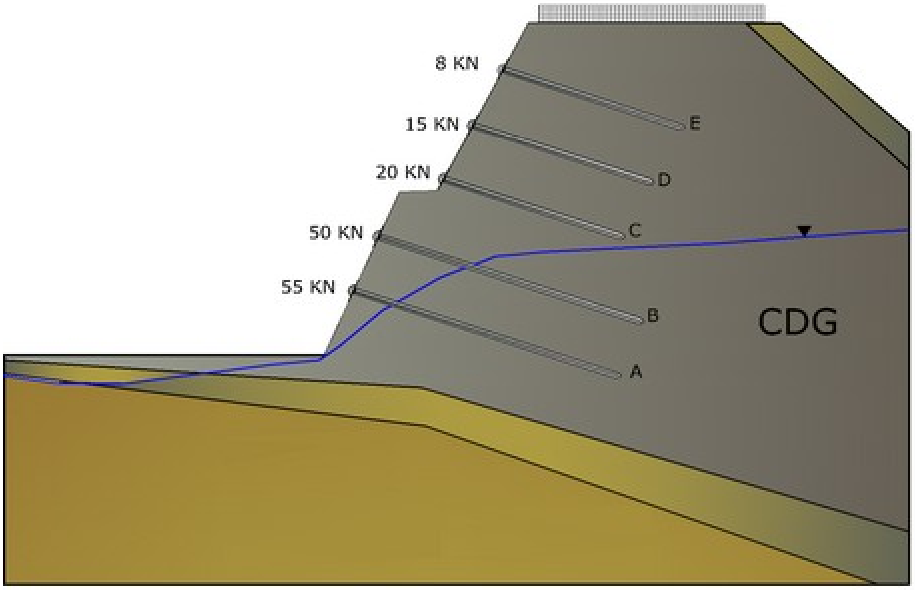
σ_{v}' = effective vertical stress of the soil calculated at the average depth of reinforcement

Φ = friction angle of the soil.

Example of calculation

Design assumptions

For the critical section of the unstable slope shown in the figure are known the following design parameters:



Soil type	**CDG (completely decomposed granite)**
C '	5 kPa,
γ	20 kN/m^3,
φ'	38°
D	0,1 m, diameter of the holes in the ground
α	15°, inclination angle of the bar
γ_w	9.81kN/m^3, specific weight of water

Nailing	Bar length (m)	Bar diameter (mm)	Horizontal distance between bars (m)	La (m)	Le (m)	Force per meter of width (KN)	Force required Tr (kN)
E	8,0	25	2	4,70	3,30	8,00	16,00
D	8,0	25	2	4,20	3,80	15,00	30,00
C	8,0	25	2	3,70	4,30	20,00	40,00
B	12,0	32	2	3,80	8,20	50,00	100,00
A	12,0	32	2	2,30	9,70	55,00	110,00

Design data

The minimum safety factors required by the regulations are given in the table:

Failure mode	Minimum safety factor (normative)
Failure due to tensile stress of the steel bar	f_{max}=0,5 fy

Failure mode	Minimum safety factor (normative)
Pullout between concrete and steel bar	3
Shear failure of adjacent soil	2

Tensile strength of the steel bar:

f_y= 460 Mpa(steel yield strength);

$\Phi \cdot f_y$= 0,5 f_y= 230 Mpa (maximum tensile stress of steel).

Maximum tensile strength of the steel bar

$$T_a = (\Phi \cdot f\ y) \cdot (d - 4)^2 \cdot \pi / 4$$

Nailing	Bar length (m)	Bar dimeter (mm)	Horizontal distance between bars (m)	Force per meter of length (KN)	Force required (KN))	Maximum allowable tensile force (KN))	Check (Ta>Tr)
E	8,0	25	2,0	8,0	16,0	79,66	ok
D	8,0	25	2,0	15,0	30,0	79,66	ok
C	8,0	25	2,0	20,0	40,0	79,66	ok
B	12,0	32	2,0	50,0	100,0	141,62	ok
A	12,0	32	2,0	55,0	110,0	141,62	ok

Calculation table of the tensile strength of the steel bar

Pull-out between steel bar and concrete

f_{cu}=32Mpa, cubic strength of mortar at 28 days,

b=0.5 for bars type 2 (deformable),

SF= 3, safety factor

Maximum allowable force between mortar and steel bar:

$[\ \beta\ (f_{cu})^{1/2}] \cdot \pi \cdot (d - 4) \cdot Le / SF$

Le= effective length of the bar,

Nailing	Bar length (m)	Bar diameter (mm)	Horizontal distance between bars (m)	Free length La (m)	Effective length (m)	Force for meter of length (KN)	Required force (KN)	Maximum allowable strength (KN)	Check (Tmax> Tr)
E	8,0	25	2,0	4,70	3,30	8,0	16,0	205,26	ok
D	8,0	25	2,0	4,20	3,80	15,0	30,0	236,36	ok
C	8,0	25	2,0	3,70	4,30	20,0	40,0	267,46	ok

B	12,0	32	2,0	3,80	8,20	50,0	100,0	680,06	ok
A	12,0	32	2,0	2,30	9,70	55,0	110,0	804,46	ok

Calculation table: Check to pull-out of steel bar from mortar

Lack of adhesion between mortar and ground

$T_f = (\pi DC' + 2DK\alpha\sigma_v' \tan\varphi) \times Le$ (Mobilized resistance between mortar and ground),

$\alpha K = 1 - (\alpha / 90)(1-Ko) = 1 - (\alpha / 90)(\sin\varphi)$, inclination factor,

Completely decomposed granite (CDG) with $K\alpha = 0.897$

$T_f = (\pi DC' + 2DK\alpha\sigma_v' \tan\varphi) \times Le = (1.571 + 0.14\sigma'_v) \times Le = (1.571 + 0.140\, \sigma'_v)$

Nailing	Resistant zone		
	Effective length in CDG layer (m) Le	Depth of the midpoint of the effective length	
		Layer CDG	
		CDG	WATER
E	3,30	3,40	0,00
D	3,80	5,30	0,00
C	4,30	7,20	0,00
B	8,20	9,70	1,40
A	9,70	9,40	3,00

Calculation table: Geometrical characteristics of steel bars

Nailing	Effective vertical stress s$_v$ (kPa)	Resistance mobilized Tf (kN)	Total resistance mobilized Tf (kN)	Force required Tr (kN)	F.O.S. Tf/Tr	Check (F.O.S.)>2
	CDG	CDG				
E	68.00	36.65	36.65	16.00	2.29	OK
D	106.00	62.45	62.45	30.00	2.08	OK
C	144.00	93.58	93.58	40.00	2.34	OK
B	180.27	220.16	220.16	100.00	2.20	OK
A	158.57	230.92	230.92	110.00	2.10	OK

Calculation table: Check for failure due to lack of adhesion between mortar and ground

1.14.5 Generic work

A generic work can be defined using the **polygon** command.

Proceed as follows::

1. Select the Polygon tool and assign the vertices.
2. After the input of the vertices, press the right mouse button.
3. In the Fill section, select the option **"Consider this polygon as a material"** and assign Material characteristics.

To move the vertices of the polygon you must use the Selection tool, move the mouse over a vertex to modify, click on the point, hold down the mouse button, move the vertex to a new position. To exit the command, press the Esc key on your keyboard.

To delete the inserted polygon selects it with the Select command and press the Delete key on your keyboard.

Through generic work a variety of cases can be represented (lenses, rigid bodies, drainage trenches, excavation, etc.).

1.14.6 Reinforced earth

Reinforced earth can be used as consolidation work; to define this type of works is required data regarding the geometrical dimensions of the work (reinforced earth high, the distance between the grids and base width); data regarding the geotechnical parameters of the filling material (specific weight, friction angle) and data regarding the strength of the reinforcing grid.
The program proposes geogrids widely used in the industry with their relative characteristics.
New types of reinforcement elements can be added using the button **New reinforcement** (at right click or at the end of the predefined list), or the user can choose (and/or modify) already defined reinforcement elements.
Each type defined, when inserted, fits to the inclination of the profile at the point of insertion, therefore, if you want to assign a particular inclination to the reinforced earth, you have to assign previously the same inclination to the segment of the profile where you want to insert the work.

The stabilizing effect of the intervention on the slope is determined by the weight of the filling material, by the frictional resistance that develops on the slices and by the tensile strength of the reinforcement.
The resistance introduced in the calculation of stability is evaluated on the "effective" length of the reinforcements, ie that part of geogrid which is not affected by the sliding surface.

1.15 Sliding surface

The analyzes can be conducted for surfaces of circular shape or for generic/free form shape. For circular surfaces the center grid must be inserted, while the free form surfaces must be assigned by points.

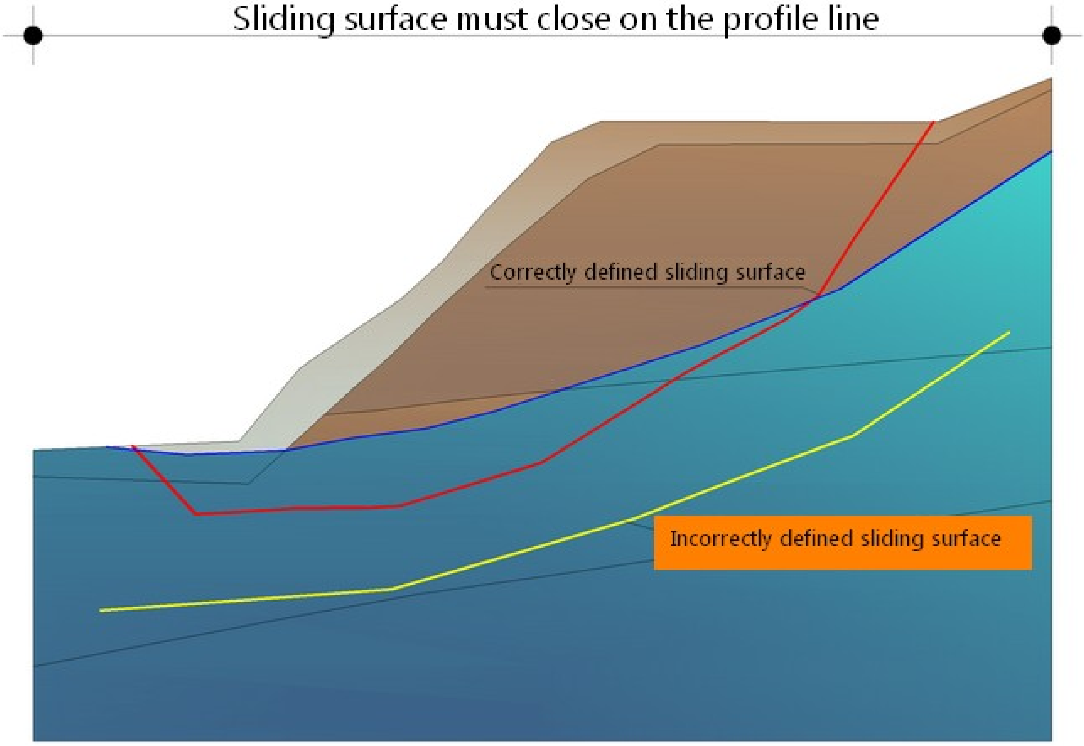
If the generic surface is chosen, the following commands are activated:

Nr. of surfaces: insert the number of generic form surfaces to analyze.
Surfaces: select the surface for which to insert the vertices.
Generate: once the first generic surface is defined, this command allows to create a number of surfaces, chosen by the user, rotated by a certain angle with respect to a chosen vertex that define the polyline of the first sliding surface.
Surface colors: assign colors to each created surface.

For more information regarding the insertion of vertices, see also Inserting vertices.

⚠ **Note:**

The sliding surface must be assigned as in the image below, otherwise the software will show a surface assignation error.



1.16 Tools

CIRCLE

Select the Circle command, move to the work area and perform a mouse click in the first insertion point, then, holding down the mouse button, go to the second insertion point of the bounding rectangle of the circle to enter, execute a click with the left mouse button and press the right button to complete the entry. The drawn circle will appear both in the final report and graphic report.

Modify circle: To modify a circle you must first select it with the Selection command, then move the mouse on the circle. Perform a click with the right button, the Circle properties window appears.

Move circle: To move a circle you must first select it with the Selection command, then move the mouse on a vertex of the rectangle that circumscribes the circle, perform a click on that point, and while holding down the mouse button, move the vertex of the rectangle to the new location. To exit the command, press the Esc key on your keyboard.

Delete circle: To delete the inserted circle selects it with the Selection command and press the Delete key on your keyboard.

LINE

Select the Line command, move to the work area and perform a mouse click in the first insertion point, then, holding down the mouse button, go to the second insertion point of the line, execute a click with the left mouse button and press the right button to complete the entry. The drawn line will appear both in the final report and graphic report.

Modify the line: To modify a line you must first select it with the Selection command, then move the mouse on the line. Perform a click with the right button of the mouse, the Line properties window appears.

Move line: To move a line you must first select it with the Selection command, then move the mouse on the point you want to modify, perform a click on that point, and while holding down the mouse button, move the vertex of the line to the new location. To exit the command, press the Esc key on your keyboard.

Delete line: To delete the inserted line selects it with the Selection command and press the Delete key on your keyboard.

POLYGON

Select the Polygon command, move to the work area and perform a mouse click in the first insertion point, then continue with a click for each vertex of the polygon and press the right button to complete the entry. The drawn polygon will appear both in the final report and graphic report.

Modify polygon: To modify a polygon you must first select it with the Selection command, then move the mouse on the vertex of the polygon you want to modify, right click on the vertex an move it to the new position holding the mouse button pressed. To exit the command, press the Esc key on your keyboard.

At right click on one of the polygon's vertices the Polygon properties window appears.

Delete polygon: To delete the inserted polygon selects it with the Selection command and press the Delete key on your keyboard.

Select the Rectangle command, move to the work area and perform a mouse click in the first insertion point, hold the mouse button pressed and draw the rectangle, release the mouse button and click to finish the insertion. The drawn rectangle will appear both in the final report and graphic report.

RECTANGLE

Modify rectangle: To modify a rectangle you must first select it with the Selection command, then move the mouse on the vertex of the rectangle you want to modify, right click on the vertex an move it to the new position holding the mouse button pressed. To exit the command, press the Esc key on your keyboard.

At right click on one of the rectangle's vertices the Rectangle (Polygon) properties window appears.

Delete rectangle: To delete the inserted rectangle selects it with the Selection command and press the Delete key on your keyboard.

TEXT

Select the Text command, move to the work area and perform a mouse click in the insertion point, hold the mouse button pressed and draw the text box, release the mouse button and click to finish the insertion. Insert the text in the window and choose also text properties from the same window. The text will appear both in the final report and graphic report.

Modify text: To modify a text you must first select it with the Selection command, then do all modifications in the Text properties window.

Delete text: To delete the inserted text selects it with the Selection command and press the Delete key on your keyboard.

RASTER IMAGES

The software allows the insertion of raster images and also offers scaling functions for them. The image displayed on the worksheet can be brought to the actual size using the **Calibrate** command, that is to say the distance measured between two points corresponds to the actual distance.

Insert: the command allows the insertion of an image, and after the image to insert is chosen, the calibration window will appear, as in the figure below:

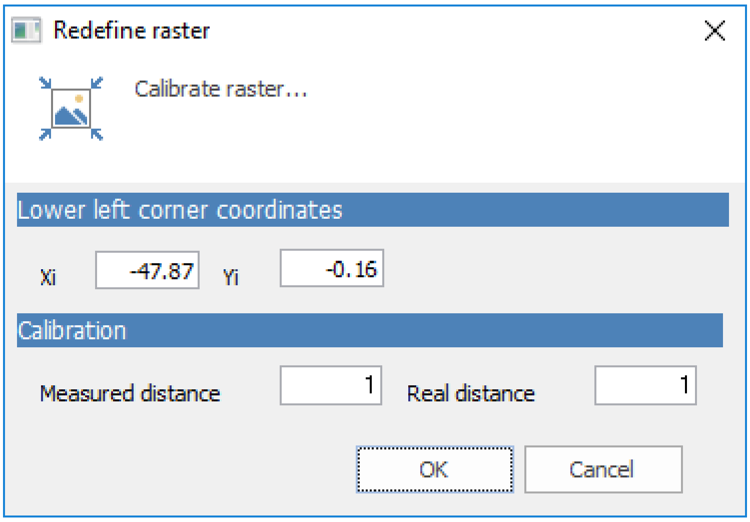


Image calibration window

This window remains in foreground to allow the user to measure, using the calibration instrument, the distance between two points in the image (to be entered in Measured distance). In Real distance must be entering the real distance between the two points.

To calibrate the image after inserting it, proceed as follows: select the image with the Selection tool, press Calibrate button in the toolbar, right-click on the image to calibrate and press Calibrate image.

Delete rasper images: select the image with the Selection tool and press Delete key on the keyboard. Select the Delete command in the toolbar to delete all raster images.

1.17 Computation

Analysis options

Choose the analysis conditions in the right panel.

Bound computation

In case of bound computation insert the required data in the right panel.

Computation methods

Choose a method to use for the computation: Fellenius, Bishop, Janbu, etc. For further information on the computation methods see also **Computation methods**.

Back Analysis

Runs the back analysis using the Janbu method. This type of analysis can be performed only for homogenous soils and for generic sliding surfaces assigned by the user. The execution of the computation returns a diagram in which cohesion and internal friction angle are reported such as to provide a safety factor equal to 1.

Perform analysis

Command that performs the calculation of stability with the method chosen by the user.

Recompute

Command that performs the calculation of the safety factor relative to a circular sliding surface already examined. To use this option, proceed as follows:

1. Choose the command Recompute from Computation menu (or click with the mouse on the Recalculate button in the right panel, or right click on the work area and choose Recompute).
2. Insert the coordinates X_0, Y_0 of the center and the value of the radius of the surface (confirm each inserted value with Enter).
3. Confirming with the Enter key the software performs the computation and shows the safety factor and the geometrical data of the examined surface.

Display safety factor

Stress diagrams

Pilings

When selecting this command a new window opens in which, for each analyzed surface is shown the insertion position of the pile, the horizontal limit load and the segment of the pile on which it is evaluated the reaction of the resistant soil with the formation of a plastic hinge at the point of intersection of the sliding surface with the pile. Obviously, such information is offered by the program only in the case where, in the definition of pilings, has been chosen as a stabilization method the limit load method of Broms or T. Ito & T. Matsui.

Dynamic Analysis

This command performs the computation in dynamic conditions. For the start of the module QSIM it is necessary to perform a first analysis under pseudo-static conditions, and, once found the surface to be examined or that with the lower safety factor found by the program, execute the command.

The opening of a dialog box will allow the user to import a design accelerogram or to generate it using the program.

The command *Dynamic analysis* starts the computation, scrolling through the calculation accelerogram, and calculates the displacements and the velocity of movement/displacement of the entire potentially instable mass.

Null displacements are associated with conditions of stability even in the presence of an earthquake that generates the considered accelerogram: in essence, the ground acceleration never exceeds the critical acceleration that triggers the movement. On the contrary, high displacements are indicative of the exceeding of mentioned acceleration, and so of unstable masses in the presence of an earthquake. For the theory used in the generation of the accelerogram you can see the help in the **QSIM module.**

1.17.1 Analysis options

Drained or undrained condition: choose the first option for an analysis in terms of effective stresses, the second in terms of total stresses. In this regard, the program will use for the computation the saturated weight and undrained cohesion cu, if is chosen the undrained analysis, and the parameters c and φ with the natural unit weight if the drained analysis is chosen.

Exclusion conditions: Excludes from the analysis those surfaces whose intersect points upstream and downstream fall in the same segment of the profile or, those for which the above intersections fall within the specified distance (exclude surfaces with intersection less than...).

Function Morgenstern and Price: For the stability analysis with the method of Morgenstern & Price is possible to choose different trends of the distribution function of the forces of the interface - it can be constant - acme - sinusoidal.

Janbu's parameter: For the analysis with the method of Janbu is permitted to assign a value to the parameter chosen by the user.

DEM method: Using the DEM method you it can be performed the stability analysis with redistribution of stresses.

1.17.2 Bound computation

All options refer to the surfaces of circular shape.

Computation bound by one point

Assigned a grid of centers, examine all the eligible surfaces passing through a point assigned by the user. To use the bound computation proceeds as follows:

1. Select Bound by one point.
2. Move the mouse over the work area.
3. Read the coordinates in the bottom left and type them in the Coordinate boxes.
4. Modify, if necessary, the coordinates of the point and the press the Apply button. Run the analysis.

Computation bound by two points

It is not necessary to assign a grid of centers, the computation takes place automatically and examine all eligible surfaces passing through the two points assigned by the user and tangent to a straight line of variable inclination between 0 ° and 90 ° in steps of 1 °. To use the bound computation proceeds as follows:

1. Select Bound by two points.
2. Move the mouse over the work area.
3. Read the coordinates in the bottom left and type them in the Coordinate boxes.
4. Modify, if necessary, the coordinates of the point and the press the Apply button. Repeat operations 3 and 4 for the second point and the run the analysis.

Computation bound by three points

It is not necessary to assign a grid of centers, the computation takes place automatically and examine all eligible surfaces passing through the three points assigned by the user. To use the bound computation proceeds as follows:

1. Select Bound by two points.
2. Move the mouse over the work area.
3. Read the coordinates in the bottom left and type them in the Coordinate boxes.

4. For each point (1, 2 and 3) the coordinates are confirmed by pressing the Apply button.

Computation bound: tangent to a straight line

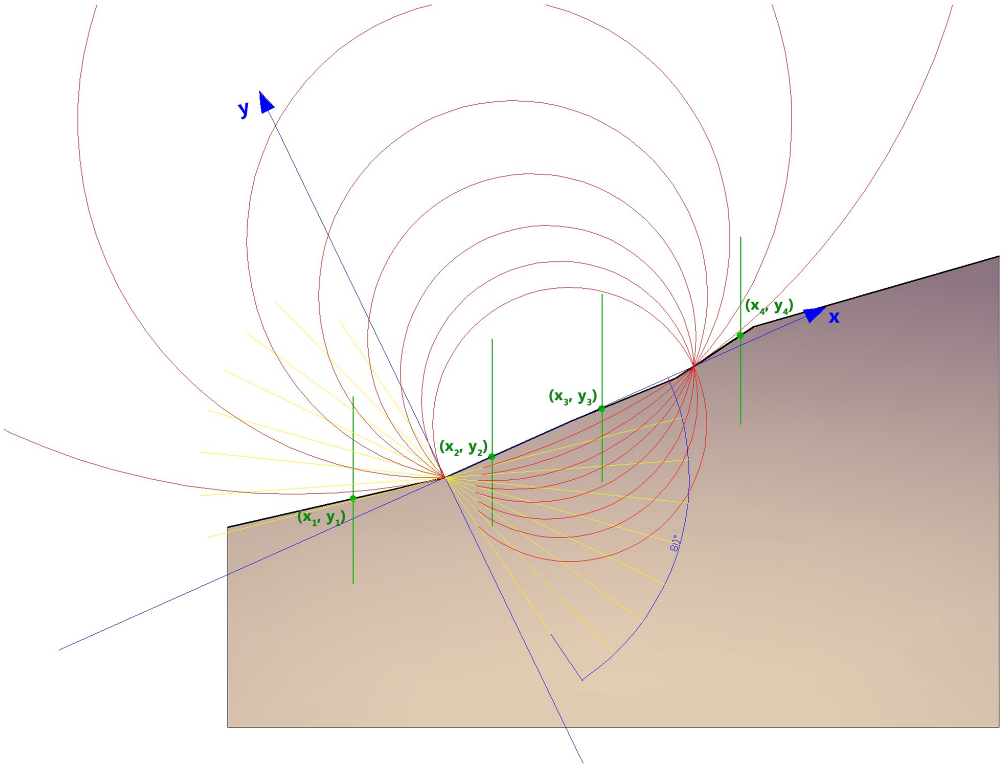
Assigned a grid of centers and a straight line is studied all eligible surfaces tangent to the line defined by the user having the center on the given grid. To use this type of bound computation proceeds as follows:

1. Select Tangent to a straight line.
2. Move the mouse over the work area.
3. Read the coordinates in the bottom left and type them in the Coordinate boxes.
4. Modify, if necessary, the coordinates of the points and the press the Apply button.

Downstream interval

Examines all surfaces whose intersection with the slope fall into two segments, one downstream and one upstream. To use this type of bound computation proceeds as follows:

1. Select Downstream interval.
2. Move the mouse over the work area.
3. Select the coordinates of the four points that define the two segments (points 1 and 2 for the downstream interval, points 3 and 4 for the upstream interval). Confirm the coordinates using the **Apply** button.
4. Run the analysis.
5. Assign the maximum angular aperture and step of the angular variation (the meaning of the two input data shown in the picture below).



In the example, the value of the maximum angle aperture is equal to 80° while the step of the angular variation is equal to approximately 10°;
in yellow are the tangent lines that bind the failure circles calculated by the program

The program does not perform the computation if it doesn't find a surface compatible with the assignments made.

Bound on a series of points

Examines a series of bound surfaces on a number of points. Insert the bounds in the appropriate grid.

1.17.3 Computation methods

The limit equilibrium method consists in the study of the equilibrium between a rigid body, such as the slope, and of a slide surface of any shape (straight line, arc of a circle, logarithmic spiral). From such equilibrium are calculated shear stresses (τ) and compared to the available strength (τ_f) calculated according to Mohr-Coulomb's failure criterion. From this comparison we derive the first indication of stability as the Safety Factor:

$$FS=\tau_f/\tau$$

Among the various equilibrium methods some consider the total equilibrium of the rigid body (Culman), while others divide the body into slices to cater for its non homogeneity and consider the equilibrium of each of these (Fellenius, Bishop, Janbu, etc.).

Slice equilibrium methods are discussed below.

Fellenius (1927)
Method valid only for sliding surfaces of circular shape, intra interface forces (between slices) are neglected. Using this method cannot be taken into consideration the works of intervention.

Bishop (1955)
Method valid only for sliding surfaces of circular shape. No contribution of the forces acting on blocks is ignored using this method that was the first to describe the problems of conventional methods.

Janbu (1956)
Janbu has extended Bishop's method to Free Form surfaces. When free form (generic form) sliding surfaces are treated the arm of the forces changes (in case of circular surfaces it is constant and equal to the radius of the arc) and therefore it is more convenient to evaluate the moment equation at the angle of each block.

Morgenstern & Price (1965)
A relation is established between the components of the interslice forces of the type $X = \lambda f(x)E$, where λ is a scale factor and f(x), a function of the position of E and X, defining a relation between the variables of the force X and the force E inside the sliding mass. The function f(x) is arbitrarily chosen (constant, sine, half-sine, trapezoidal, etc.) and has little influence on the result, but it should be verified that the values obtained for the unknowns are physically acceptable.

Abstract of article in Canadian Geotechnical journal (2002):
Slope-stability problems are usually analyzed using a variety of limit equilibrium methods of slices. When evaluating the stability conditions of soil slopes of simple configuration, circular potential slip surfaces are usually assumed and the Ordinary method (Fellenius 1936) and the simplified Bishop method (Bishop 1955) are

commonly used, the latter being preferred due to its high precision. However, in many situations, the actual failure surfaces are found to deviated largely from circular shape or the potential slip surfaces are predefined by planes of weakness in rock slopes. In such cases, a number of methods of slices can be used to accommodate the non-circular shape of slip surfaces (Janbu 1954; Lowe and Karafiath 1960; Morgenstern and Price 1965; Spener 1967; U.S. Army Corps of Engineers 1967; and etc.), among which the Morgenstern-Price method (Morgenstern and Price 1965) is regarded as the most popular because it satisfies complete the equilibrium conditions and involves the least numerical difficulties. The basic assumption underlying the Morgenstern-Price method is that the ratio of normal to shear interslice forces across the sliding mass is represented by an interlace force function that is the product of a specified function f(x) and an unknown scaling factor l. According to the vertical and force equilibrium conditions for individual slices and the moment equilibrium condition for the whole sliding mass, two equilibrium equations are derived involving the two unknowns the factor of safety Fs and the scaling factor l, thereby rendering the problem determinate. Unfortunately, solving for Fs and l is very complex since the equilibrium equations are highly nonlinear and in rather complicated form. Some sophisticated iterative procedures (Morgenstern and Price 1967; Fredlund and Krahn 1977; Chen and Morgenstern 1983; Zhu et al. 2001) have been developed for such purposes. Although these procedures can give converged solutions to Fs and l, they are not easily accessible to general geotechnical designers who have to rely on commercial packages as a black box.

Spencer (1967)

The interface forces along the division surfaces of the individual slices are oriented parallel to one another and inclined with respect to the horizontal by an assigned angle.

Bell (1968)

The equilibrium is obtained by equating to zero the sum of the horizontal forces, the sum of the vertical forces and the sum of the moments with respect to the origin. Distribution functions of normal stresses are adopted.

Sarma (1973)

The method of Sarma is a simple, but accurate method for the analysis of slope stability, which allows to determine the horizontal seismic acceleration required so that the mass of soil, delimited by the sliding surface and by the topographic profile, reaches the limit equilibrium state (critical acceleration Kc) and, at the same time,

allows to obtain the usual safety factor obtained as for the other most common geotechnical methods.
It is a method based on the principle of limit equilibrium of the slices, therefore, is considered the equilibrium of a potential sliding soil mass divided into n vertical slices of a thickness sufficiently small to be considered eligible the assumption that the normal stress Ni acts in the midpoint of the base of the slice.

Zeng Liang (2002)

Zeng and Liang carried out a series of parametric analyzes of a two-dimensional model developed by finite element code, which reproduces the case of drilled shafts (piles immersed in a moving soil).
The two-dimensional model reproduces a slice of soil of unit thickness and assumes that the phenomenon occurs in plane strain conditions in the direction parallel to the axis of the piles.
The model was used to investigate the influence on the formation of an arch effect of some parameters such as the distance between the piles, the diameter and the shape of the piles, and the mechanical properties of the soil. The authors identify the relation between the distance and the diameter of the piles (s/d), the dimensionless parameter determining the formation of the arch effect.
The problem appears to be statically indeterminate, with a degree of indeterminacy equal to (8n-4), but in spite of this it is possible to obtain a solution by reducing the number of unknowns and thus taking simplifying assumptions, in order to make determined the problem.

Numerical method of displacements

D.E.M. Discrete Element Method (1992)

With this method, the soil is modeled as a series of discrete elements, which we will call "slices", and takes into account the mutual compatibility between the slices. A slope in the present model is treated as comprised of slices that are connected by elastoplastic Winkler springs. One set of springs is in the normal direction to simulate the normal stiffness. The other set is in the shear direction to simulate the sliding resistance at the interface. The behavior of the normal and shear springs is elastoplastic. The normal springs do not yield in compression, but they yield in tension, with a small tensile capacity for cohesive soil (tension cutoff) and no tensile capacity for frictional soil.

The computationl methods and the different theories are also given in the final report.

Slope calculates with only one method at a time. However, the computation method can be changed and then used the command Recompute in "***Computation summary***" to recalculate the same surface.

1.17.4 Computation summary

This tabbed panel shows the salient results of the calculation and enables the display of results to be tailored to requirement.

Recompute: performs the computation of the safety factor relative to a circular sliding surface already examined. To use this option proceeds as follows:

1. Select the surface to be recalculated with the command Display safety factor from the Computation menu, press the ESC key to confirm the position of the surface.
2. Insert the coordinates X_C, Y_C of the center and the value of the surface radius (confirm each entered value with Enter).
3. By pressing Recompute the software performs the computation and displays the factor of safety and the geometrical data of the examined surface.

Recompute is very useful because it allows to change the computation method, to insert works and to examine the surface calculated previously.

Display intervals: It is possible to divide by color the surfaces whose safety factor falls in value ranges defined by the user. The discretization of the intervals can occur automatically is chosen the option in the panel at the bottom right.

The user can customize the intervals by choosing the lower and upper limits (ex. 0-1,3) and the corresponding color or define a gradient of colors. Personalized choice of intervals is performed, after deselecting the option *Automatic selection for display color intervals* and selecting *Interval color selection* option.

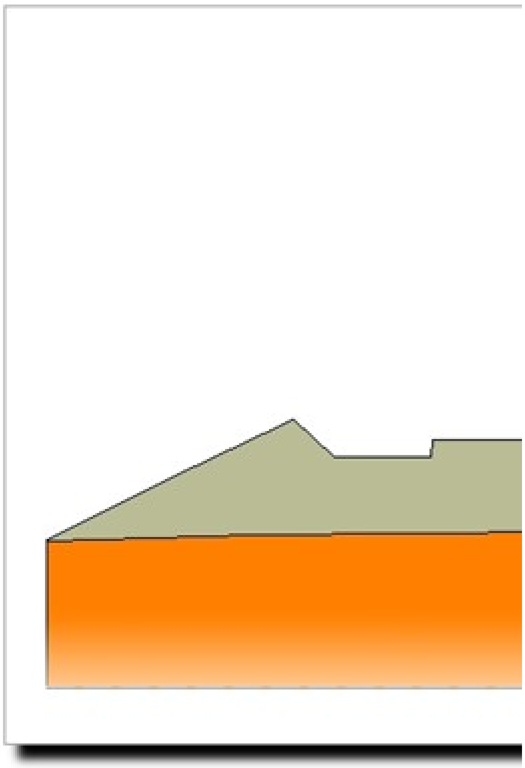
View

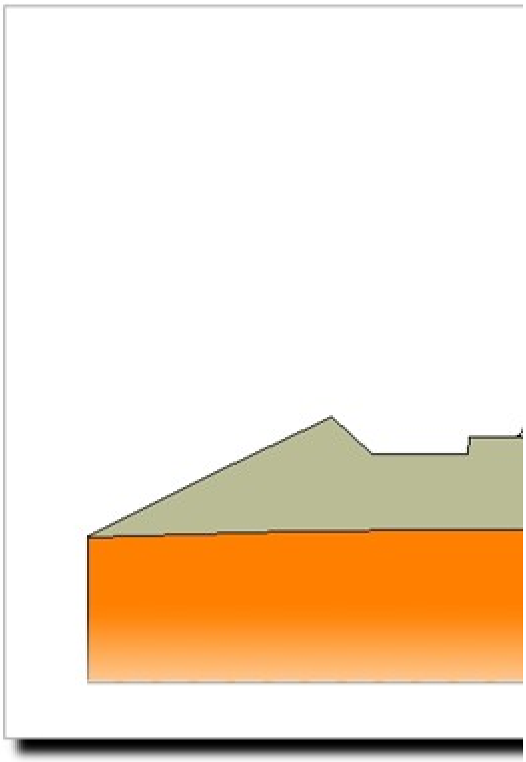
Centers grid: Displays the grid center chosen by the user.
Factors map: Displays on the centers' grid the safety factors related to each center.
Color map: Displays the map of the factors in colors. This option is useful to determine if the chosen grid of centers verifies all possible sliding surfaces compatible with the position of the grid and the geometry of the slope. The presence of bands of color is well defined, indicates the correct positioning of the mesh; on the contrary, strong dispersions of color must direct the user to the choice of a different grid.
Isolines: Displays on the grid the curves connecting the points with the same safety factor.

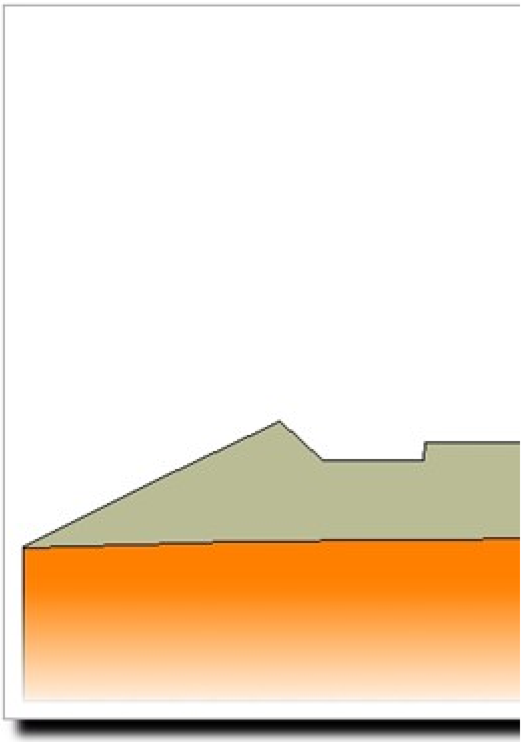
1.17.5 Display safety factor

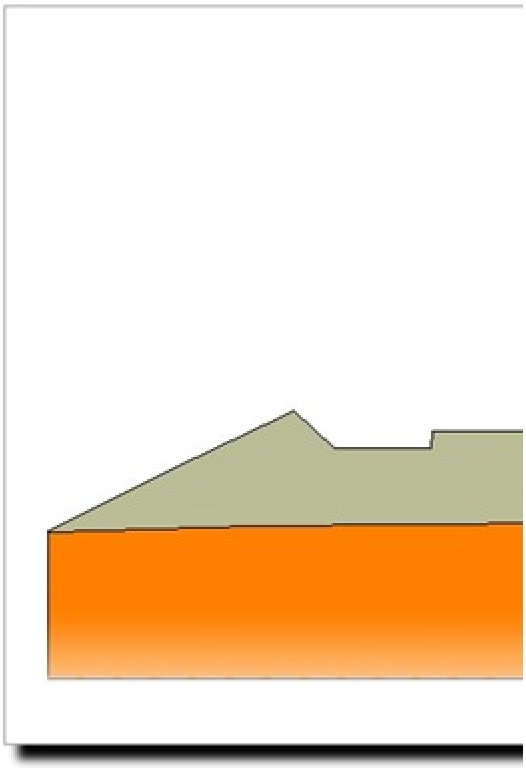
Computation option that can only be selected if it is enabled the circular sliding surface option. To use this option, proceed as follows:

1. After running the automatic computation select, in *Computation menu*, select the command *Display safety factor*.
2. Move the mouse over the grid of centers.
3. On the status bar will show the safety factor corresponding to each surface of radius R_C and center (X_C, Y_C).
4. Quit the command by pressing the ESC key.







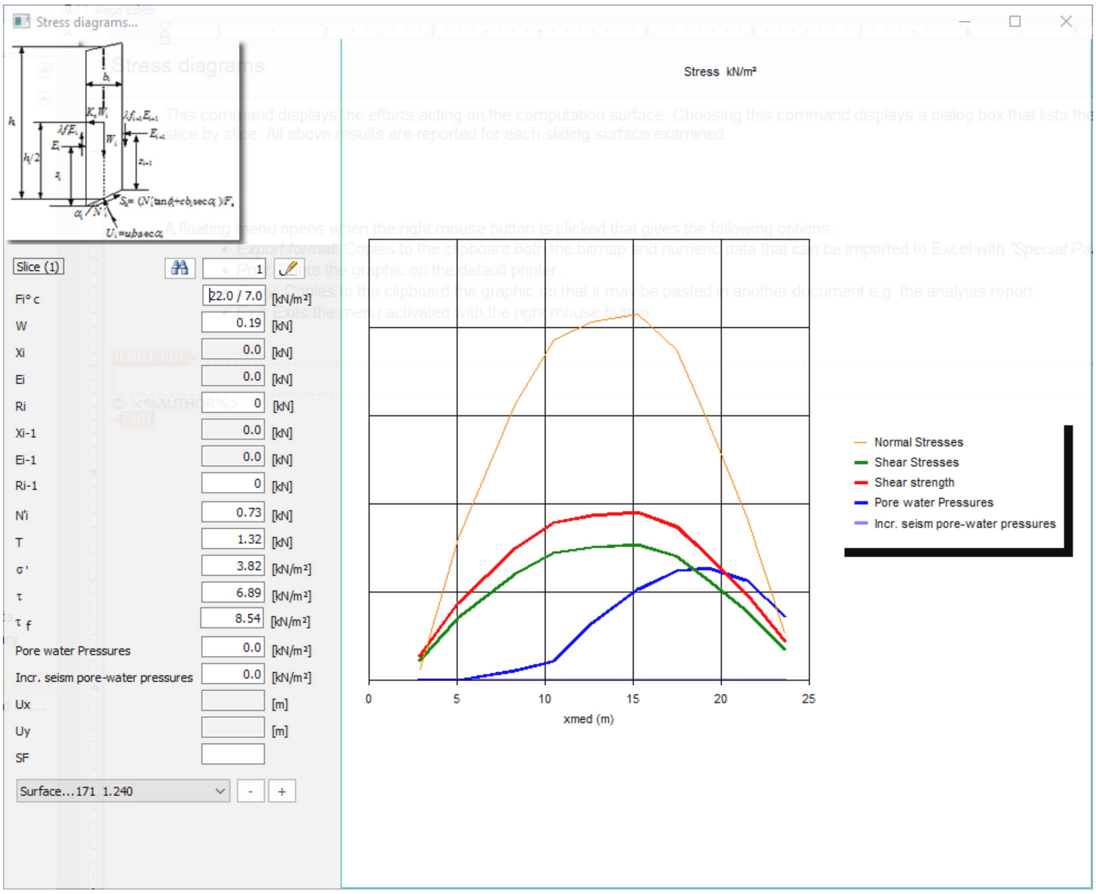


Note:

The surface interrogated by the user is stored on the screen once you exit the command; the surface can also be printed.

1.17.6 Stress diagrams

This command displays the efforts acting on the computation surface. Choosing this command displays a dialog box that lists the trend of normal and shear stresses along the surface and, on the left side of the window, all efforts slice by slice. All above results are reported for each sliding surface examined.



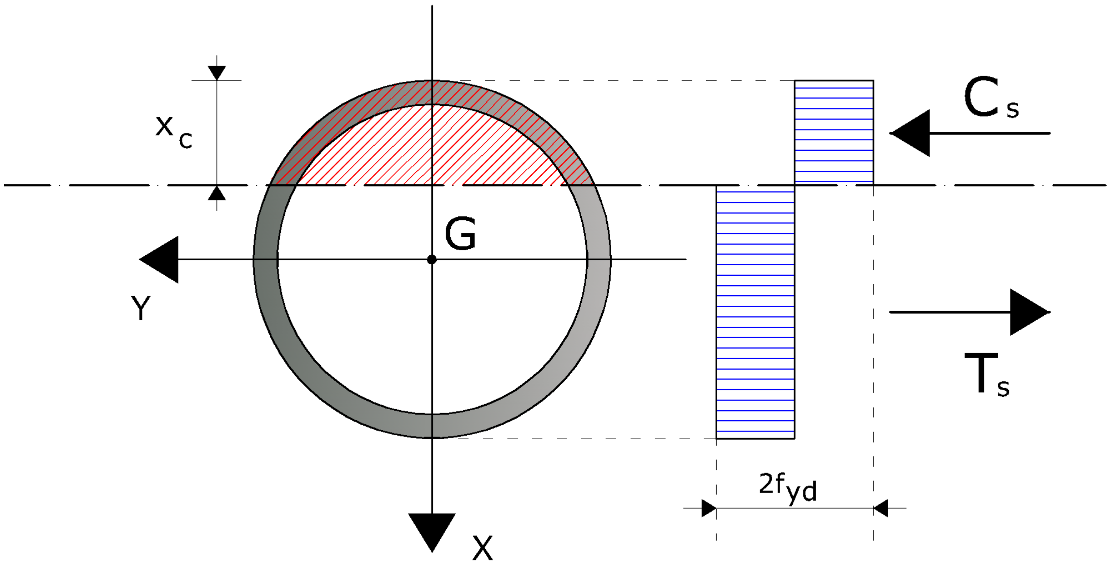
A floating menu opens when the right mouse button is clicked that gives the following options:

- *Export format*: Copies to the clipboard both the bitmap and numeric data that can be imported to Excel with '*Special Paste*' command.
- *Print*: Prints the graphic on the default printer.
- *Copy*: Copies to the clipboard the graphic so that it may be pasted into another document e.g. the analysis report.
- *Exit*: Exits the menu activated with the right mouse button.

1.18 Computation of the Yield Moment

Calculation of the Yield Moment for a steel tubular section

The section under examination is the following:



Reference diagram for the calculation of the yield moment for a steel tubular section

The calculation of the yield moment has been made assuming, for steel, a rigid-plastic constitutive bound, with limit yield stress equal to f_{yd}. The field moment has been determined by interpolation on the section interaction curve.

In order to construct the interaction curve of the section, the following procedure has been followed:

- step 1 Fixing the depth of the neutral axis (x_c) - (starting from x_c = 0);

- step 2 Calculating the resultant in terms of normal stress (Nd);

- step 3 Calculating the resultant moment (Md) as regards the geometric center of gravity of the section;

- step 4 Memorizing the calculated point (Nd, Md);

- step 5 Increasing x_c, if x_c is still lower than or equal to the diameter of the section, then back to step 1, otherwise the procedure is over.

> ⚠ **Note:**
>
> In this way, the upper part of the interaction domain is constructed. In any case the lower part is identical, but heme symmetrical.

The generic point of the interaction domain has been calculated using the following formulas:

$$Nd = Ac\_s(x_c) \cdot fyd - At\_s(x_c) \cdot fyd$$

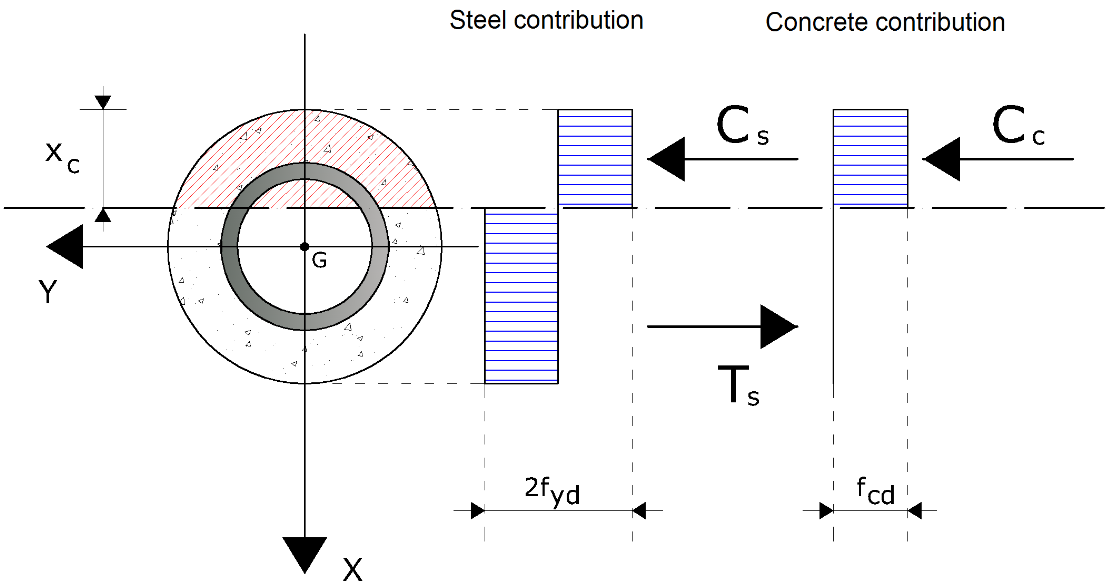
$$Md = Ac\_s(x_c) \cdot fyd \cdot dCs + At\_s(x_c) \cdot fyd \cdot dTs$$

In the previous formulas, the symbols have the following meaning:

- **Ac_s** Area of compressed steel;
- **At_s** Tensile stress steel area;
- **fcd** concrete compressive cylinder strength;
- **fyd** steel yield strength;
- **dCs** Distance between the resultant of the compression stresses of steel and the center of gravity of the section;
- **dTs** Distance between the resultant of the tensile stresses of steel and the center of gravity of the section.

Calculation of the Yield Moment for a steel tubular section immersed into a concrete circular section

The previous formulation, used for the tubular section, can be extended to the case in which the tubular section is immersed into a concrete section. In this case, it is necessary to take into account the concrete contribution, according to the following diagram:



The diagram for the calculation of the field moment for a steel tubular section immersed in a circular concrete section

As you can observe, the concrete type considered as a reactant is only compressed concrete. The values of the stresses at a fixed depth of the neutral axis of the section is as follows:

$$Nd = Ac\_s(x_c) \cdot fyd + Ac\_c(x_c) \cdot fcd - At\_s(x_c) \cdot fyd$$
$$Md = Ac\_s(x_c) \cdot fyd \cdot dCs + Ac\_c(x_c) \cdot fcd \cdot dCc + At\_s(x_c) \cdot fyd \cdot dTs$$

In the previous formulas, the symbols have the following meaning:

- **Ac_s** Area of compressed steel;
- **Ac_c** Area of compressed concrete;
- **At_s** Tensile stress steel area;
- **fcd** concrete compressive cylinder strength;
- **fyd** steel yield strength;

- **dCs** Distance between the resultant of the compression stresses of steel and the center of gravity of the section;
- **dCc** Distance between the resultant of the compression stresses if concrete and the center of gravity of the section;
- **dTs** Distance between the resultant of the tensile stresses of steel and the center of gravity of the section.

Calculating the yield moment for a circular RC section

In this case too a constitutive bond of the rigid-plastic materials is assumed, with limit stresses equal to fcd and fyd for concrete and steel, respectively. The reference diagram is the following:

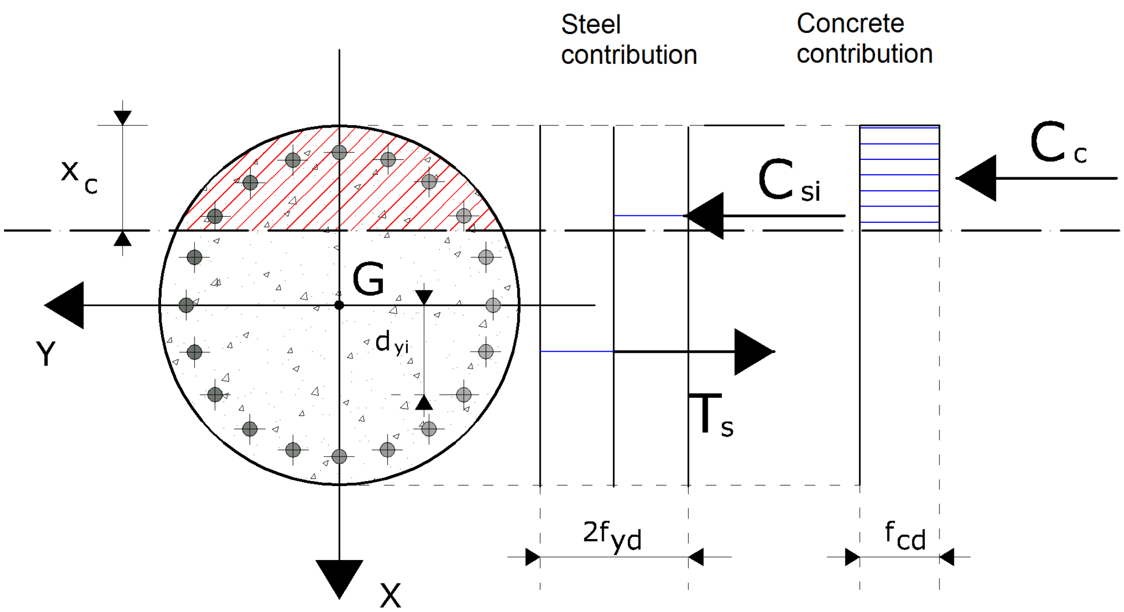


Diagram for the calculation of the field moment of a circular RC section

In this case, the value of the stresses – in correspondence of a preset depth of the neutral axis – is the following:

$$Md = \sum_{i=1}^{i=nb} Asi \cdot fyd \cdot dyi + Ac\_c(x_c) \cdot fcd \cdot dCc$$

In the previous formulas, the symbols have the following meaning:

- **Ac_c** Area of compressed concrete;
- **Asi+** Area of the i-the reinforcement bar situated above the neutral axis;
- **Asi-** Area of the i-the reinforcement bar situated under the neutral axis;
- **Asi** Area of the i-the reinforcement bar;
- **fcd** concrete compressive cylinder strength;
- **fyd** steel yield strength;
- **dCc** Distance between the resultant of the compression stresses if concrete and the center of gravity of the section;
- **dyi** is the positive distance (along the vertical) measured between the center of gravity of the i-th reinforcing bar and the center of gravity of the section

1.19 Pore water pressure

⚠ **Note:**

Attention, for the computation of pore water pressure **additional data in geotechnical properties** must be assigned.

Pore pressure after the earthquake
To calculate pore pressures ***after the earthquake***, assign 0 to horizontal and vertical seismic coefficients, and a value different from 0 for the seismic acceleration.

Shear strength under seismic loading
In the absence of appropriate experimental determinations, obtained from laboratory cyclic tests, the reduction of shear strength in conditions of seismic loading can be estimated using empirical relations of literature, as indicated in the following

paragraphs, with reference to the case of the analysis conducted in terms of effective stress or in terms of total stresses.

Analysis in terms of effective stress

The increase in pore pressure must be assessed in the case of saturated soils if the shearing induced by the seismic action is greater than the value of the volumetric strain, γ_v.

In partially saturated soils, the pore pressure increases during the application of the seismic activity, but generally is maintained below atmospheric pressure; in this case, can be assumed a zero value of the pore pressure for the entire period of application of the load (σ'=s) and the analysis can be performed using the characteristics of resistance determined in drained tests carried out on specimens of the same material previously saturated. For the calculation of Du should be distinguished the behavior of the soil in relation to their different nature, cohesive or noncohesive.

Cohesive soils

In cohesive soils, the increase in pore pressure Δu, at a certain depth, can be estimated by the following empirical relation (Matsui et al., 1980):

$$\frac{\Delta u}{\sigma'_0} = \beta \cdot \left[\log \left\{ \frac{\gamma_{c,\max}}{\gamma_v} \right\} \right]$$

Where σ'_0 is the initial value of the effective average pressure at the considered depth, $\gamma_{c,max}$ is the maximum shearing strain reached during the earthquake and β=0.45 is an experimental coefficient. The volumetric strain γ_v, determinable with cyclic laboratory tests, can be calculated in first approximation by the relation:

$$\gamma_v = A \cdot (OCR - 1) + B$$

Where OCR is the over-consolidation ratio, A and B are experimental coefficients that, in the absence of a direct determination, may be derived as a function of the plasticity index; view table below:

Ip (%)	A	B
20	$0.4\ 10^{-3}$	$0.6\ 10^{-3}$
40	$1.2\ 10^{-3}$	$1.1\ 10^{-3}$
55	$2.5\ 10^{-3}$	$1.2\ 10^{-3}$

Values suggested for A and B coefficients

The value of $\gamma_{c,max}$ related on the considered depth can be determined by an analysis of the local seismic response. Alternatively, it is determined preliminarily the value of τ_{max} using the empirical relation:

Where a_{max}, expressed in g, is the peak acceleration at the ground level on the vertical relative to the considered point; g is the gravitational acceleration; σ_v is the total vertical stress; r_d is a reduction factor which takes account of the seismic action at the depth of interest which leads into account the deformability of the subsoil. The factor r_d may be calculated, to a first approximation, with the following expression:

$$r_d = 1 - 0.015 \cdot z$$

Where z is the depth at the considered point. The maximum shear deformation induced by the earthquake is obtained then by the relation:

$$\gamma_{c,\max} = \frac{\tau_{\max}}{G}$$

Where the shear modulus G can be determined, using an iteration process, from the curve (G-γ) obtained from laboratory tests.

Granular soils

In granular soils, the increase in pore pressure generated by seismic activity can be estimated by the following empirical relation (Seed & Booker, 1997):

$$\frac{\Delta u_N}{\sigma'_0} = \frac{2}{\pi} \sin^{-1}\left\{\left(\frac{N}{N_L}\right)^{\frac{1}{a}}\right\}$$

Where Δu_N is the increment of pore pressure after N cycles of load, σ'_0 is the initial value of the effective average pressure at the considered depth, N is the number of load cycles of constant amplitude equivalent to the earthquake and N_L is the number of load cycles required to produce the liquefaction in the soil. The experimental constant a can be calculated using the relation proposed by Fardis & Veneziano (1981) as a function of the relative density D_r (in fraction):

The term ε_q has log-normal distribution with average and unit variance equal to 0.1. To determine the number of cycles N that appears in one of the previous relations it is necessary to approximate the history of irregular shear deformation induced by the earthquake with an equivalent cyclic stress of constant amplitude (τ_{eq}) and equivalent number of cycles (N_{eq}) following one of the many procedures described in the literature. For example, using the procedure proposed by Biondi et al. (2004) we obtain:

$$\tau_{eq} = 0.65 \cdot \tau_{\max}$$

$$N_{eq} = e^{\left(\alpha + \beta \cdot \ln(a\max) + \gamma \cdot \ln(Ia) + \delta \cdot \ln(v0) + \varepsilon \cdot \ln\left(T^D\right)\right)}$$

In the first of the above equations τ_{max} is the maximum shear stress induced by the earthquake at a considered depth, whose value can be estimated by an analysis of the local seismic response or, in the first approximation by the relation used in the section on cohesive soils. In the second of the above equations the various terms have the following meaning:

- I_a is the intensity of Arias (m/s);
- v0 is the intensity of the intersections with the time axis of the accelerogram (s-1);
- T^D is the duration of the accelerogram defined by Trifunac and Brady (s);

The intensity of Arias is defined by the following formula:

$$I_a = \frac{\pi}{2 \cdot g} \cdot \int_0^\infty [a(t)]^2 dt$$

The other symbols that appear, ie α - β - γ - δ - ε, are constants for which are is recommended the following values:

$$\begin{cases} \alpha = -1.629 \\ \beta = -2.493 \\ \gamma = 1.239 \\ \delta = 0.854 \\ \varepsilon = -0.307 \end{cases}$$

For the determination of the value of N_L we can refer to methods that are based on graphic interpolations, or may be used the results of cyclic triaxial tests or cyclic simple shear.

1.19.1 Reduction in undrained strength

Analysis in terms of total stresses

Cohesive soils

If the analysis is performed in terms of total stresses, the value of the undrained cohesion cu must be reduced compared to the static case to take account of the degradation consequent to the cyclical nature of seismic stresses. Is generally neglected in favor of safety, the possible increase of the undrained strength, which can manifest in cohesive soils of high plasticity due to the high speed of application of the loads. An estimate of the reduction coefficient of the undrained strength, δ_{cu} can be obtained using the equation:

$$\delta_{cu} = N^{-t}$$

Where N is the number of load cycles induced by the earthquake and t is a parameter of degradation that can be estimated with the following relation:

$$t = s \cdot (\gamma_c - \gamma_v)^r$$

The function of the cyclic shear strain γ_c and of the volumetric strain, the last one assessed as outlined above. The values of S and r can be estimated as a function of the plasticity index, PI and the over-consolidation ratio OCR as reported in the following table:

	OCR=1			OCR=2	OCR=4
	PI=15	*PI=30*	*PI=50*	*PI=50*	*PI=50*
s	0.195	0.095	0.075	0.054	0.042
r	0.600	0.600	0.495	0.480	0.423

Coefficients for the calculation of the cyclic degradation (Matasovic, 1993)

The number of N cycles can be evaluated by calculating the number of intersections with the time axis in the time interval between the first and last exceeding of a predetermined acceleration threshold (usually equal to 0.05 g). For the cyclic shear strain γ_c can be used the following relation:

In which the value of the shear modulus G is determined by an iterative process from the curve (G-γ) obtained from laboratory tests and τ_{eq} is calculated with the formula discussed earlier.

1.19.2 Calculation of the shear modulus

Calculation of the shear modulus G

The shear modulus to be introduced in the above equations can be calculated by referring to diagrams of the type of those shown in the figure below, in which the trend of the shear modulus is a function of shear strain and of plasticity index of the soil PI.

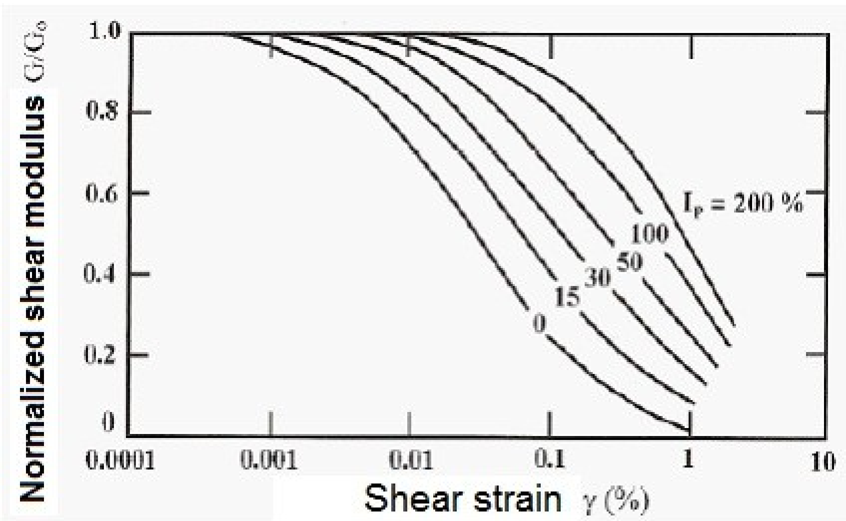


Diagram used for the calculation of the shear modulus

As can be observed to be plotted is not directly G but rather the ratio G/G_0 where G_0 is the dynamic low strain shear modulus. The modulus G_0 can be derived by correlating it with the S-waves velocity of the layer:

Where r is the mass density of the soil created by the unit weight divided by the gravitational acceleration in m/s^2 ($9.81\ m/s^2$). Alternatively, there are several formulations for the evaluation of G_0, including the following:

Method of Imai and Tomauchi

This method correlates the dynamic low strain shear modulus with the average peak strength:

$$G_0 = 28 \cdot q_c^{0.611} \left[\frac{kg}{cm^2} \right]$$

Where q_c is the average peak strength in the layer measured with the static penetrometer. The result is expressed in kg/cm^2.

Method of Ohsaki & Iwasaki

This method becomes valid as part of clean sand or plastic sand. This method correlates the low strain shear modulus with the number of blows in the layer and with the average particle size (granulometry) of the ground:

$$G_0 = a \cdot Nspt^{b} \left[\frac{t}{m^2} \right]$$

Where N_{spt} is the average number of blows of the layer and the constants a and b are shown in table below:

a	b	Granulometry
650	0.94	Clean sand
1182	0.76	Plastic sands

Values ∂f parameters used in the formula of Ohsaki & Iwasaki

1.19.3 Computation of NL

Calculation of the number of cycles to obtain the soil liquefaction

Can be evaluated with the aid of diagrams of the type of those in the figure below. The trend of N_L is a function of the amplitude of the shear stress imposed τ_{hv} (Normalized compared with the initial value of the average effective stress) and for different values ∂f the relative density Dr of the soil.

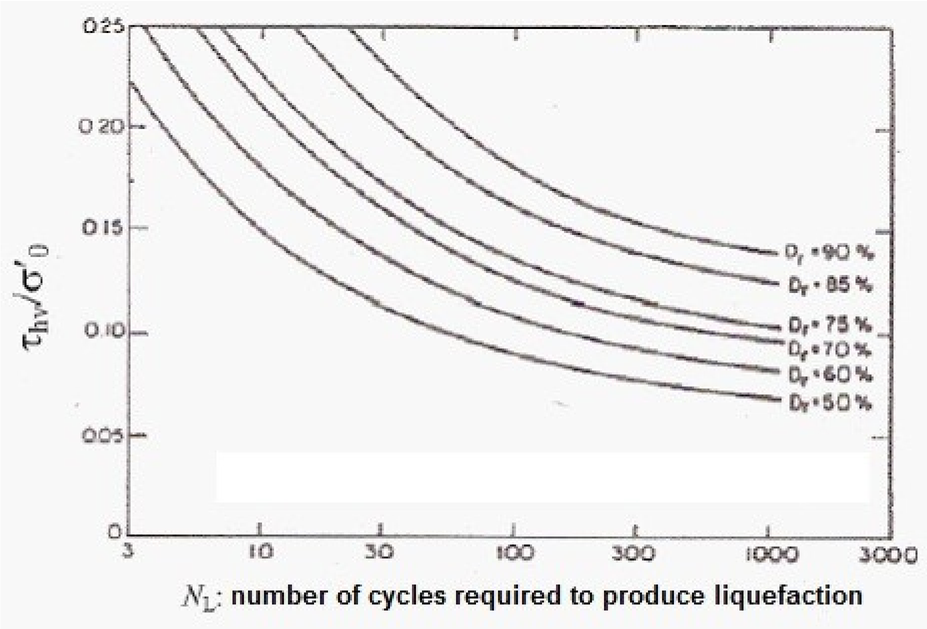


Diagram of reference for the calculation of N_L

1.19.4 Accelerogramm integration

Time conversion factor

A conversion factor which multiplies the time contained in the accelerogram file. It is necessary to convert the time into seconds.

Acceleration conversion factor

A conversion factor which multiplies the acceleration contained in the accelerogram file. It is necessary to convert the acceleration into m/s^2.

Separator used in the file

The separator used in the accelerogram file to divide the acceleration column from the time column.

Open

Import the accelerogram file.

Parameters

Arias intensity [Ia]

A parameter representing the index of intensity and frequency of the seismic waves. It is defined, less one constant, as the integral of the square of the accelerogram (extended to the whole duration of the earthquake).

Intensity of the intersections of the accelerogram with the time axis [ν_0].

It is calculated as the ratio of the number of times the accelerogram intersects the time axis to the duration of the seismic event.

Duration of the seismic motion [TD]

The duration of the seismic motion defined by *Trifunac* (Trifunac & Brady 1975). It is calculated as the time interval elapsing between the attainment of 5% of Ia and 95% of Ia (Ia stands for Arias intensity).

Loaded accelerogram

On the loaded accelerogram, a scale factor which only affects its visualization is activated.

Calculation of the accelerogram integration parameters

Accelerogram parameters

The study of the issue relevant to the evaluation of the increase in the pore pressure in soils, in case of a seismic action, requires the calculation of some parameters aimed at identifying the frequency and intensity properties of the accelerogram. The parameters to be determined are the following:

- Arias intensity (Ia in m/s);
- Intensity of the intersections of the accelerogramm with the time axis (ν_0 in 1/s);
- Actual duration of the motion defined by Trifunac (Trifunac and Brady, 1975, TD in s);

1. Arias intensity

Arias intensity is a parameter relevant to the accelerogram which provides information on the intensity and frequency of it. This parameter is defined according to the following ratio:

where

- T_{MAX} represents the whole duration of the accelerogram;
- *a(t)* represents the accelerogram

As a rule, the values of this parameter vary between 0.05 and 2.5/3.

2. Intensity of the intersections with the time axis

This parameter is defined though the following formula:

$$\nu_0 = \frac{Ni}{T_{MAX}}$$

where

- Ni is the number of times – throughout the duration of the accelerogram – the acceleration intersects the time axis;
- T_{MAX} is the duration of the accelerogram.

3. Actual duration according to Trifunac

This parameter is used to identify the time interval elapsing between the following extreme cases:

$$tds = t : I_A(tds) = 5\% I_A$$
$$tde = t : I_A(tde) = 95\% I_A$$

where

$$I_A(t*) = \frac{\pi}{2 \cdot g} \int_0^{t*} [a(t)]^2 \cdot dt$$

According to the previously provided definitions, the time defined by Trifunac is equal to:

$$TD = tde - tds$$

1.20 Theoretical notes

The resolution of a stability problem requires taking into account field equations and constitutive laws. The first ones are equilibrium equations, the second ones describe the behavior of the soil. These equations are particularly complex since the soil is of

multiphase systems, which can be traced to single-phase systems only in dry soil conditions, or analysis in drained conditions.
In most cases we have to deal with a material that if is saturated is at least two-phase, which makes the treatment of the equilibrium equations considerably complicated. Furthermore, it is practically impossible to define a constitutive law of general validity, because the soils have a non-linear behavior already at small strains, are anisotropic, and also their behavior depends not only on the deviatoric stress but also on the normal stress. Because of the above difficulties are introduced simplifying assumptions:

- Simplified constitutive laws are used: rigid, perfectly plastic model. It is assumed that the resistance of the material is expressed solely by the parameters cohesion (c) and angle of shearing resistance (φ'), constant for the soil and characteristic of the plastic state; therefore it is assumed valid the Mohr-Coulomb failure criterion:

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

where:

τ = shear strength, with the size of a stress;
c' = cohesion;
u = pore-water pressure;
φ' = shearing resistance angle.

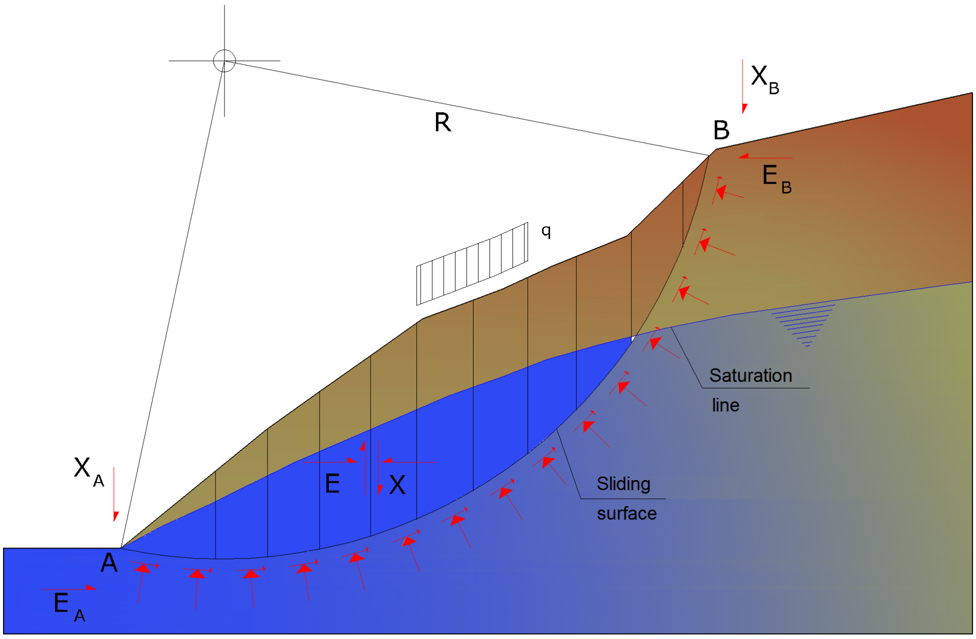
- In some cases are met only in part the equations of equilibrium.

1.20.1 Limit equilibrium method (LEM)

The limit equilibrium method consists of the study of the equilibrium between a rigid body, such as the slope, and of a slip surface of any shape (straight line, arc of a circle, logarithmic spiral, etc.). From this equilibrium are calculated shear stresses (τ) and compared to the available resistance (τ_f) calculated according to Mohr-Coulomb's failure criterion. From this comparison we derive the first indication of stability as the Factor of Safety:

$$\mathbf{F} = \tau_f / \tau$$

Among the various equilibrium methods some consider the global equilibrium of the rigid body *(Culman)*, while others divide the rigid body into slices to cater for its non homogeneity and consider the equilibrium of each of these *(Fellenius, Bishop, Janbu, etc.)*.



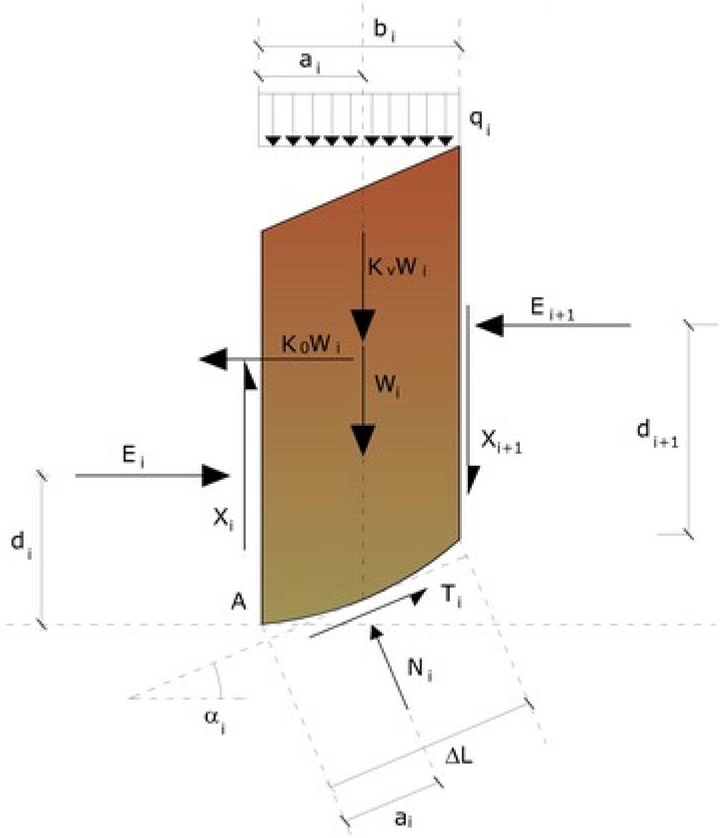
Representation of a calculation section of a slope

Slice method

The volume affected by slide is subdivided into a convenient number of slices. If the number of the slices is *n* the problem presents the following unknowns:

- *n* values of normal forces N_i acting on the base of each slice;
- *n* values of shear forces at the base of slice T_i;
- *(n-1)* normal forces E_i acting on slice interface;
- *(n-1)* tangential forces X_i acting on slice interface;
- *n* values of the coordinate "*a*" that identifies the point of application of E_i;
- *(n-1)* values of the coordinate that identifies the point of application of X_i;
- an unknown constituted by the safety factor F.

In all the unknowns are *(6n-2)*.



Actions on the i-the slice

While the equations are:

- Equilibrium equations of the moments *n*;
- Equilibrium equations at the vertical displacement *n*;
- Equilibrium equations at the horizontal displacement *n*;
- Equations relative to the failure criterion *n*.

In all there are 4*n* equations.
The problem is statistically indeterminate to the extent of:

$$i = (6n-2)-(4n) = 2n-2$$

The degree of indetermination is further reduced to *(n-2)* as it is assumed that N_i is applied at a mid point of the slice, which is equivalent to assume the hypothesis that total normal stresses are distributed uniformly.
The various methods that are based on limit equilibrium theory differ in the way in which the *(n-2)* degrees of indetermination are eliminated.

1.20.1.1 Fellenius method (1927)

With this method (only valid for circular form slide surfaces) interslice forces are ignored and thereby the unknowns are reduced to:

- *n* values of normal forces N_i;
- *n* values of shear forces T_i;
- *1* Safety factor.

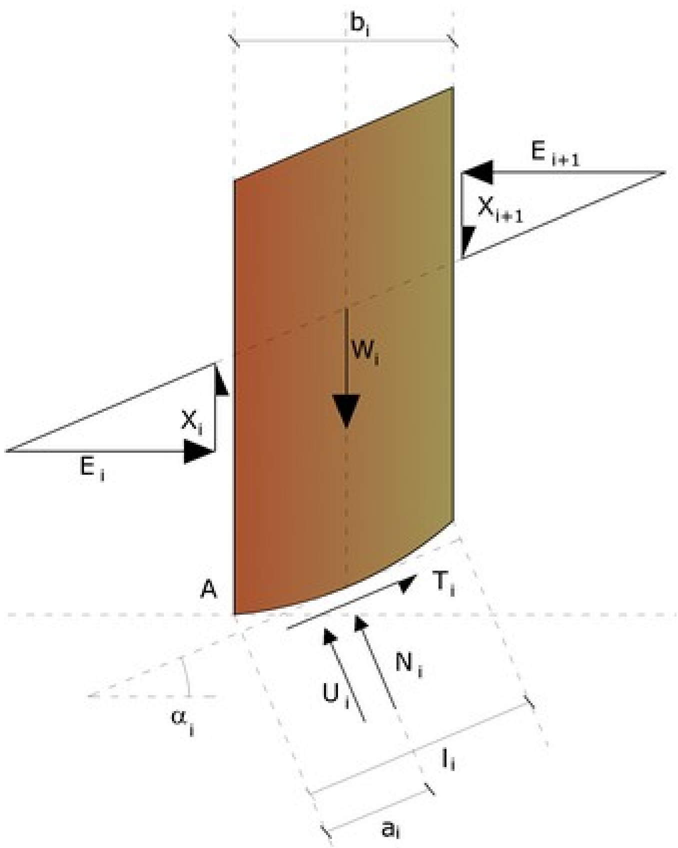
Unknowns are *(2n+1)*

The available equations are:

- *n* equilibrium equations at the vertical displacement;
- *n* equations relative to the failure criterion;

1 equation of the global moments.

$$F = \frac{\sum \{c_i \times l_i + (W_i \times \cos\alpha_i - u_i \times l_i) \times \tan\varphi_i\}}{\sum W_i \times \sin\alpha_i}$$



Actions on the i-the slice according to Fellenius

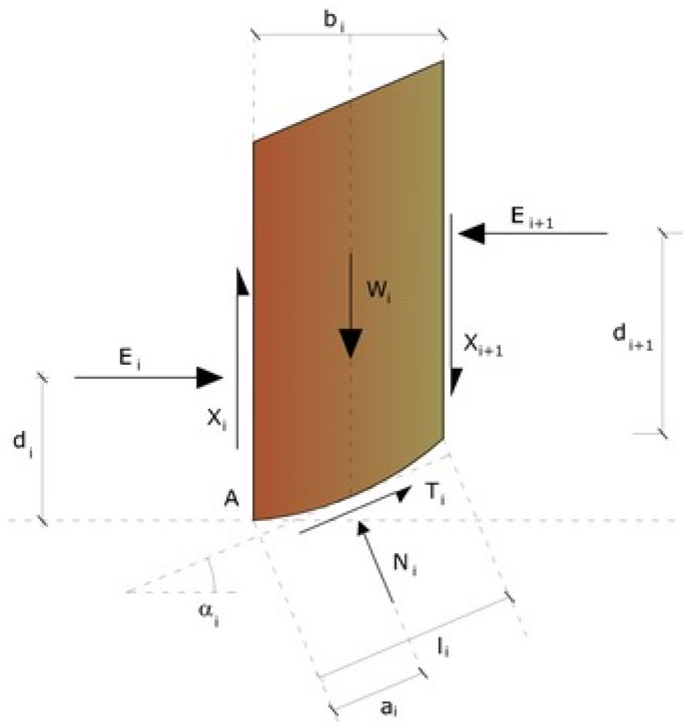
This equation is easy to solve, but it was found that provides conservative results (low safety factors) especially for deep surfaces or at the increase in the value of the pore pressure.

1.20.1.2 Bishop method (1955)

None of the contributing forces acting on slices is ignored using this method that was the first to describe the problems of conventional methods.
The equations used to resolve the problem are:

$\Sigma F_V = 0$, $\Sigma M_0 = 0$, failure criterion.

$$F = \frac{\Sigma \left\{ c_i \times b_i + (W_i - u_i \times b_i + \Delta X_i) \times tan\ \varphi_i \right\} \times \dfrac{\sec \alpha_i}{1 + \tan \alpha_i \times \tan \varphi_i / F}}{\Sigma W_i \times \sin \alpha_i}$$



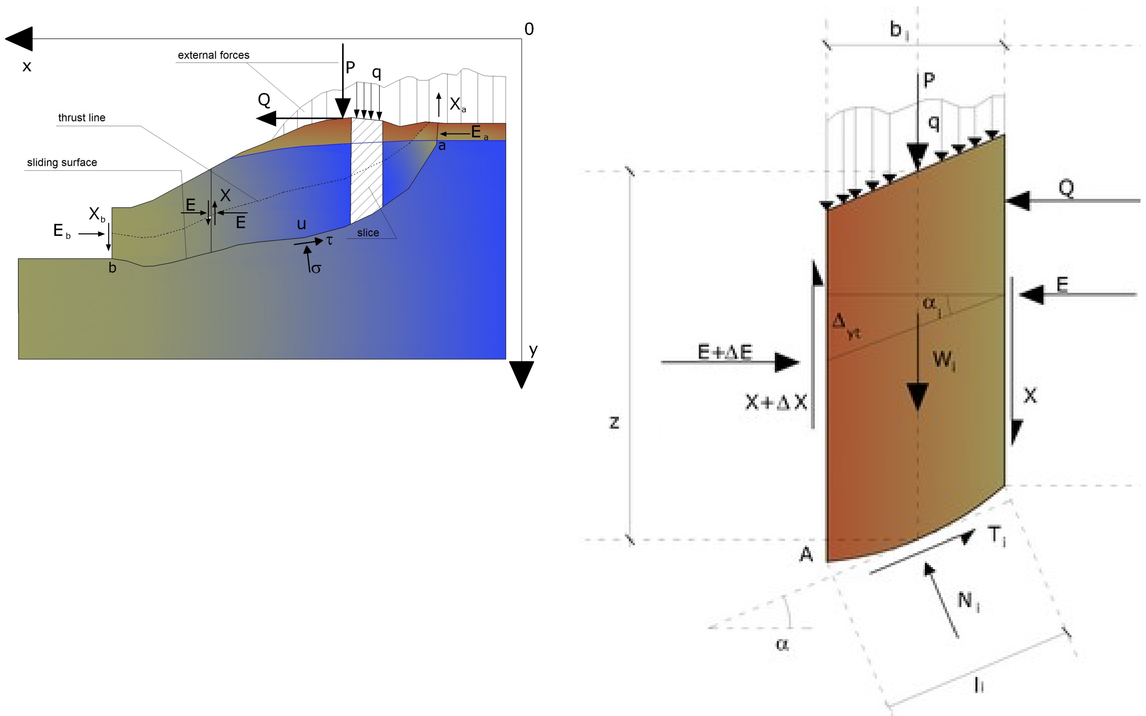
Actions acting on the i-the slice according to Bishop (ordinary method)

The values of F and ΔX for each item that satisfy this equation give a rigorous solution to the problem. As a first approximation should be taken $\Delta X = 0$ and iterate for calculating the factor of safety, this procedure is known as the ordinary method of Bishop, the errors made compared to the complete method are about 1%.

1.20.1.3 Janbu method (1967)

Janbu has extended Bishop's method to free form surfaces. When free form (generic form) sliding surfaces are treated, the arm of the forces changes (in case of circular surfaces it is constant and equal to the radius of the arc) and therefore it is more convenient to evaluate the moment equation at the angle of each slice.
With the method of Janbu are taken into account the forces of interaction between the slices, but they are deemed to act along a predetermined thrust line. The solution is obtained by subsequent iterations.

$$\mathrm{F} = \frac{f_0 \cdot \Sigma\left\{\mathrm{c_i} \times b_i + (\mathrm{W_i} - \mathrm{u_i} \times b_i + \Delta \mathrm{X_i}) \times \tan\varphi_i\right\} \times \dfrac{F}{\cos^2\alpha_i \cdot (1 + \tan\alpha_i \times \tan\varphi_i)}}{\Sigma \mathrm{W_i} \times \tan\alpha_i}$$

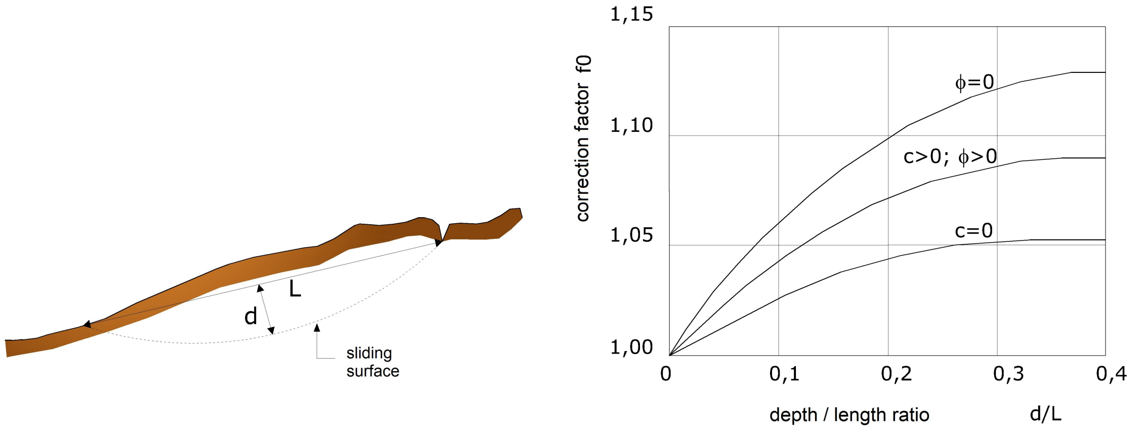


Actions on the i-the slice according to Janbu and representation of the whole slice

Assuming ΔXi = 0 is obtained by the ordinary method.

Janbu also proposed a method for the correction of the safety factor obtained by the ordinary method according to the following:

$$F_{corretto} = f_0\, F$$

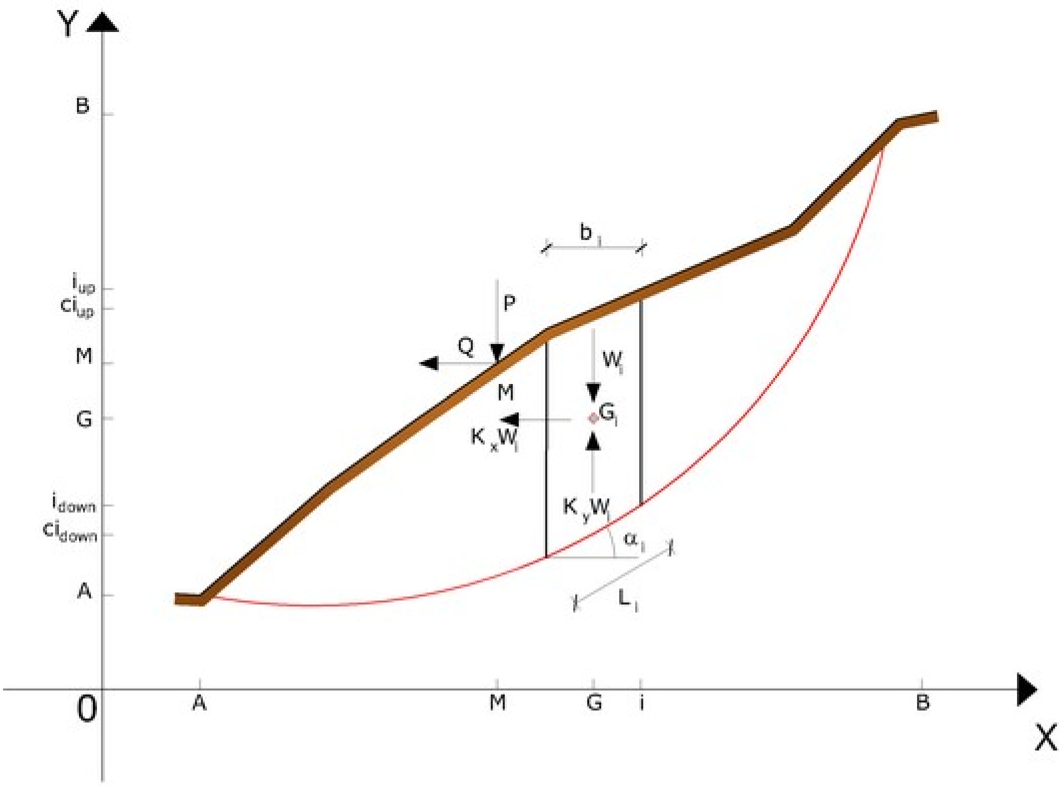


Calculation of the correction factor f_0

where f_o , empirical correction factor, depends on the shape of the sliding surface and on the geotechnical parameters.
This correction is very reliable for slightly inclined slopes.

1.20.1.4 Bell method (1968)

The forces acting on the mass that slides include the weight of soil, W, seismic horizontal and vertical pseudo static forces K_xW e K_YW, the horizontal and vertical forces *X* and *Y* applied externally to the slope profile, the resultant of the total normal and shear stresses σ and τ acting on the potential sliding surface.
The total normal stress can include an excess of the pore pressure u that must be specified with the introduction of the parameters of effective force.
In practice, this method can be considered as an extension of the method of the circle of friction for homogeneous sections previously described by *Taylor*.



Representation on the Cartesian plane of the block and of the forces acting on the i-the slice

In accordance with Mohr-Coulomb theory in terms of effective stress, the shear stress acting on the base of the i-the slice is given by:

$$T_i = \frac{c_i \cdot L_i + (N_i - \mu_{ci} \cdot l_I)\tan\varphi_i}{F}$$

where:

F = safety factor;

c_i = the effective cohesion (or total) at the the base of the i-the slice;

φ_i= the effective friction angle (= 0 with the total cohesion) at the the base of the i-the slice;

L_i = the length of the base of the i-the slice;

μ_{ci} =the pore pressure at the center of the base of the i-the slice.

The equilibrium is obtained by equating to zero the sum of the horizontal forces, the sum of the vertical forces and the sum of the moments compared to the origin.

Is adopted the following assumption on the variation of the normal stress acting on the potential sliding surface:

$$\sigma_{ci} = \left[C_1 (1 - Ky) \cdot \frac{w_i \cdot \cos \alpha_i}{L_i} \right] + C_2 f\left(x_{ci}, y_{ci}, z_{ci}\right)$$

in which the first term of the equation includes the expression:

$W_i \cos \alpha_i / L_i$ = total value of the normal stress associated with the ordinary method of the slices.

The second term of the equation includes the function:

$$f = \sin 2\pi \left(\frac{x_n - x_{ci}}{x_n - x_0} \right)$$

Where x_0 and x_n are respectively the abscissa of the first and the last point of the sliding surface, while x_{ci} represents the abscissa of the midpoint of the base of the i-the slice.

A sensitive part of weight reduction associated with a vertical acceleration of the ground K_y g can be transmitted directly to the base and this is included in the factor $(1 - K_y)$.

The total normal stress at the base of a slice is given by:

$$N_i = \sigma_{ci} \cdot L_i$$

The solution of the equations of equilibrium is obtained by solving a linear system of three equations obtained by multiplying the equilibrium equations for the safety factor F, substituting the expression of N_i and multiplying each term of the cohesion by an arbitrary factor C_3.

It is assumed a linear relation between the coefficient, determined using Cramer's rule, and the safety factor F. The corrected value of F can be obtained from the linear interpolation formula:

where the numbers in parentheses (1) and (2) indicate initial and subsequent values ∂f the parameters F and C_3.

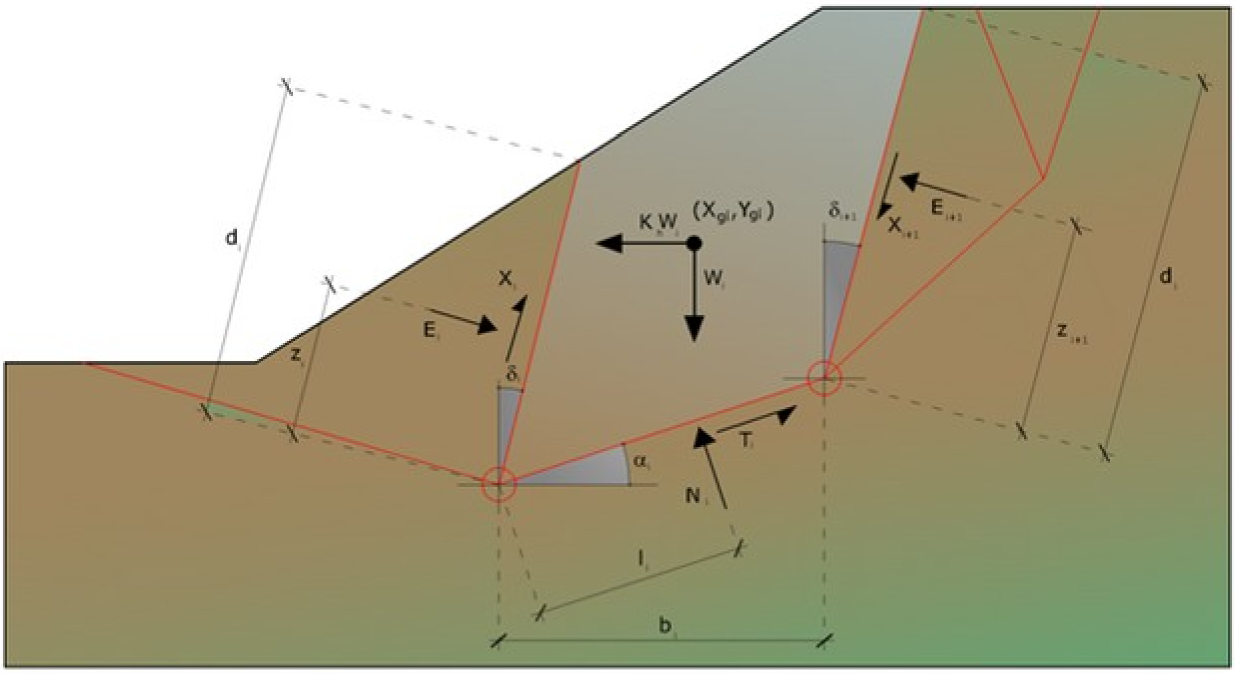
Any pair of values ∂f the safety factor in the close to a physically reasonable estimate can be used to initiate an iterative solution.

The required number of iterations depends on the initial estimate and the desired accuracy of the solution, normally, the process converges rapidly.

1.20.1.5 Sarma method (1973)

The method of Sarma is a simple, but accurate method for the analysis of slope stability, which allows determining the horizontal seismic acceleration required so that the mass of soil, delimited by the sliding surface and by the topographic profile, reaches the limit equilibrium state (critical acceleration K_c) and, at the same time, allows obtaining the usual safety factor obtained as for the other most common geotechnical methods.

It is a method based on the principle of limit equilibrium of the slices, therefore, is considered the equilibrium of a potential sliding soil mass divided into n vertical slices of a thickness sufficiently small to be considered eligible in the assumption that the normal stress N_i acts in the midpoint of the base of the slice.



Actions on the i-the slice according to Sarma

The equations to be taken into consideration are:

- The equation of equilibrium to the horizontal translation of the single slice;
- The equation of equilibrium to the vertical translation of the single slice;
- The equation of equilibrium of moments.

Equilibrium conditions to horizontal and vertical translation:

$$N_i \cos \alpha_i + Ti \sin \alpha_i = W_i - \Delta X_i$$
$$T_i \cos \alpha_i - Ni \sin \alpha_i = KW_i + \Delta E_i$$

It is also assumed that in the absence of external forces on the free surface of mass occurs:

$$\Sigma \Delta E_i = 0$$
$$\Sigma \Delta X_i = 0$$

where E_i and X_i represent, respectively, the horizontal and vertical forces on the face of the generic **i** slice.
The equilibrium equation of moments is written choosing as a reference point the center of gravity of the entire mass; so that, after performing a series of trigonometric and algebraic positions and transformations, in Sarma's method the solution of the problem passes through the resolution of two equations:

$$* \sum \Delta X_i \cdot tg(\Psi'_i - \alpha_i) + \sum \Delta E_i = \sum \Delta_i - K \cdot \sum W_i$$

$$** \sum \Delta X_i \left[(y_{mi} - y_G) \cdot tg(\psi'_i - \alpha') + (x'_i - x_G) \right] = \sum W_i \cdot (x_{mi} - x_G) + \sum \Delta_i \cdot (y_{mi} - y_G)$$

But the solution approach, in this case, is completely inverted: the problem in fact makes it necessary to find a value of K (seismic acceleration) corresponding to a given safety factor; and in particular, find the acceleration value K corresponding to the safety factor F = 1, that is, the critical acceleration.

Therefore:

$K = K_c$ critical acceleration if F = 1

F = Fs safety factor in static conditions if K = 0

The second part of the problem of Sarma's method is to find a distribution of internal forces X_i and E_i such as to verify the equilibrium of the slice and the global equilibrium of the whole body, without breach of the failure criterion.
It has been found that an acceptable solution of the problem can be achieved by assuming the following distribution for the X_i forces:

$$\Delta X_i = \lambda \cdot \Delta Q_i = \lambda \cdot \left(Q_{i+1} - Q_i\right)$$

where Q_i is a known function, in which are taken into consideration the average geotechnical parameters on the i-the face of the slice i, and λ represents an unknown.

The complete solution of the problem is obtained therefore, after a few iterations, with the values ∂f K_c, l and F, that allow to obtain even the distribution of interslice forces.

1.20.1.6 Spencer method (1967)

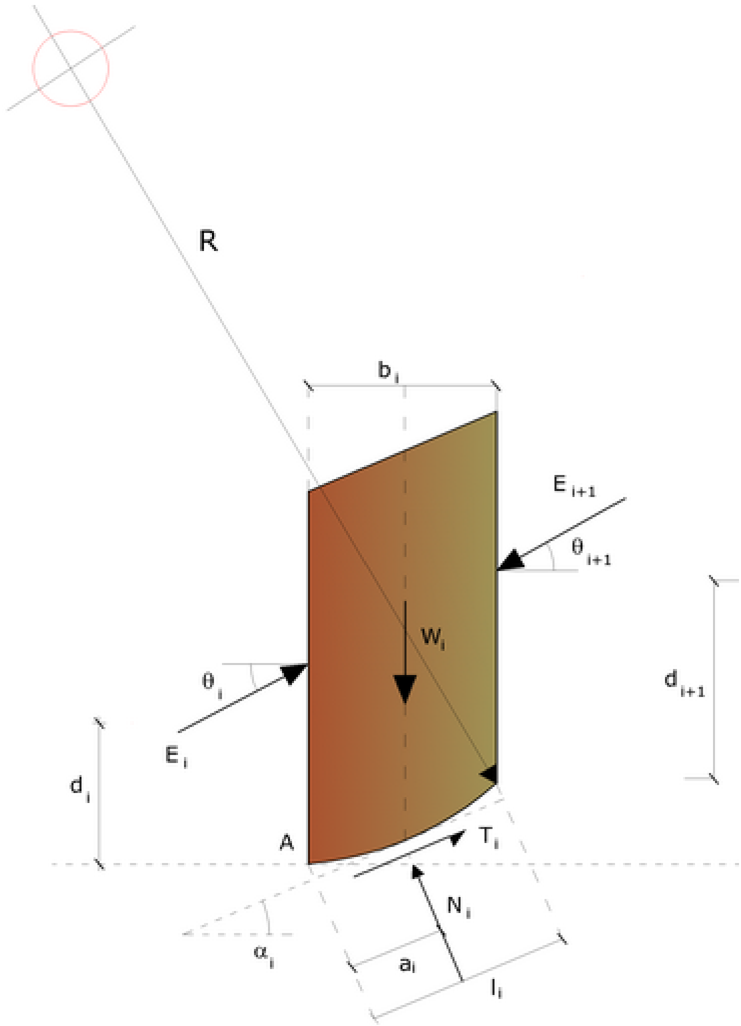
The method is based on the following assumptions:

The interslice forces along the division surfaces of the individual slices are oriented parallel to one another and inclined compared to the horizontal by an angle q, all moments are null M_i =0 i=1.....n

Basically, the method satisfies all the statics equations and is equivalent to the method of Morgenstern and Price when the function f(x) = 1.

Imposing the equilibrium of moments from the center of the arc described by the sliding surface:

$$\sum Q_i R \cos(\alpha - \theta) = 0 \qquad (1)$$



Actions on the i-the slice according to Spencer

where:

$$Q_i = \frac{\frac{c}{F_s}\left(W\cos\alpha - \gamma_w hl\sec\alpha\right)\frac{tg\alpha}{F_s} - Wsen\alpha}{\cos(\alpha-\beta)\left[\frac{F_s + tg\varphi \cdot tg(\alpha-\theta)}{F_s}\right]}$$

interaction force between the slices;

R = radius of the arc of the circle;

θ = angle of inclination of the force Q_i from the horizontal.

Imposing the equilibrium of horizontal and vertical forces we have respectively:

$$\sum(Q_i \cos\theta) = 0 \; ; \; \sum(Q_i sen\theta) = 0 \qquad (2)$$

With the assumption of Q_i forces parallel to each other, we can also write:

$$\sum Q_i = 0$$

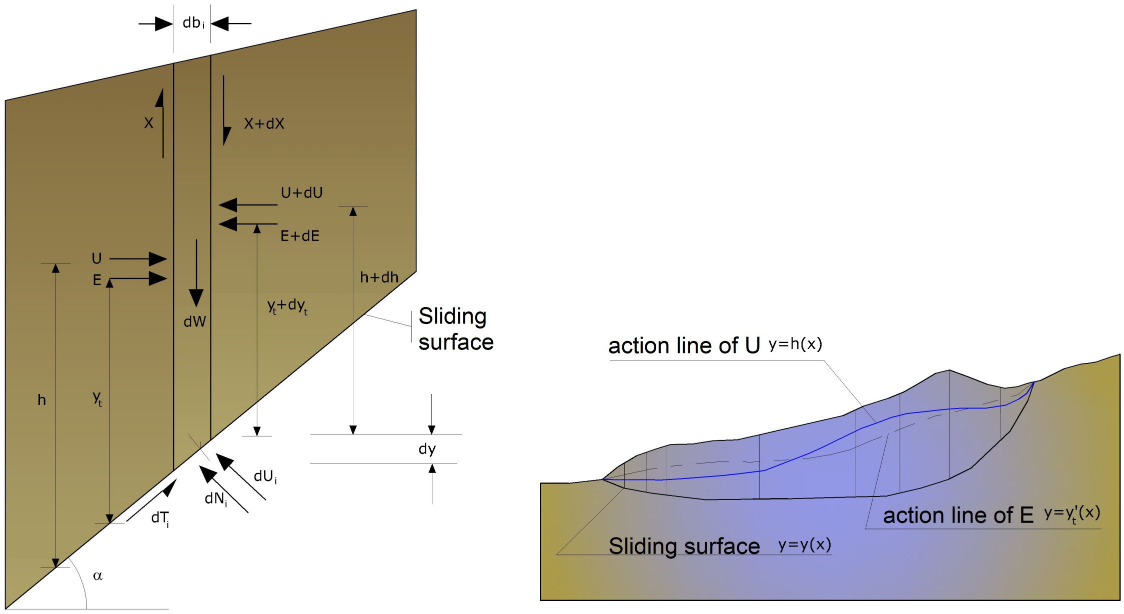
The method proposes to calculate two safety factors: the first (F_{sm}) obtainable from 1), related to the equilibrium of moments, the second (F_{sf}) obtainable from 2) related to the equilibrium of forces. In practice, we proceed by solving 1) and 2) for a given range of values of the angle θ, considering as a unique value of the safety factor that for which

$$F_{sm} = F_{sf}.$$

1.20.1.7 Morgenstern & Price method (1965)

A relation is established between the components of the interslice forces of the type X= λ f(x) E, where λ is a scale factor and f(x), a function of the position of E and X, defining a relation between the variation of the force X and the force E inside the sliding mass. The function f(x) is arbitrarily chosen (*constant, sine, half-sine, trapezoidal*, etc.) and has little influence on the result, but it should be verified that the values obtained for the unknowns are physically acceptable.

The particularity of the method is that the mass is divided into infinitesimal strips to which are imposed the equations of equilibrium to the horizontal and vertical translation and failure on the basis of the strips themselves. This leads to a first differential equation that binds interslice unknown forces E, X, the factor of safety Fs, the weight of the infinitesimal strip dW and the resultant of the pore pressure at the base dU.



Actions on the i-th slice according to Morgenstern & Price add representation of the entire mass

The result is the so-called "*equation of forces*":

$$c'\sec^2\frac{\alpha}{F_s}+tg\varphi'\left(\frac{dW}{dx}-\frac{dX}{dx}-tg\alpha\frac{dE}{dx}-\sec\alpha\frac{dU}{dx}\right)=$$

$$=\frac{dE}{dx}-tg\alpha\left(\frac{dX}{dx}-\frac{dW}{dx}\right)$$

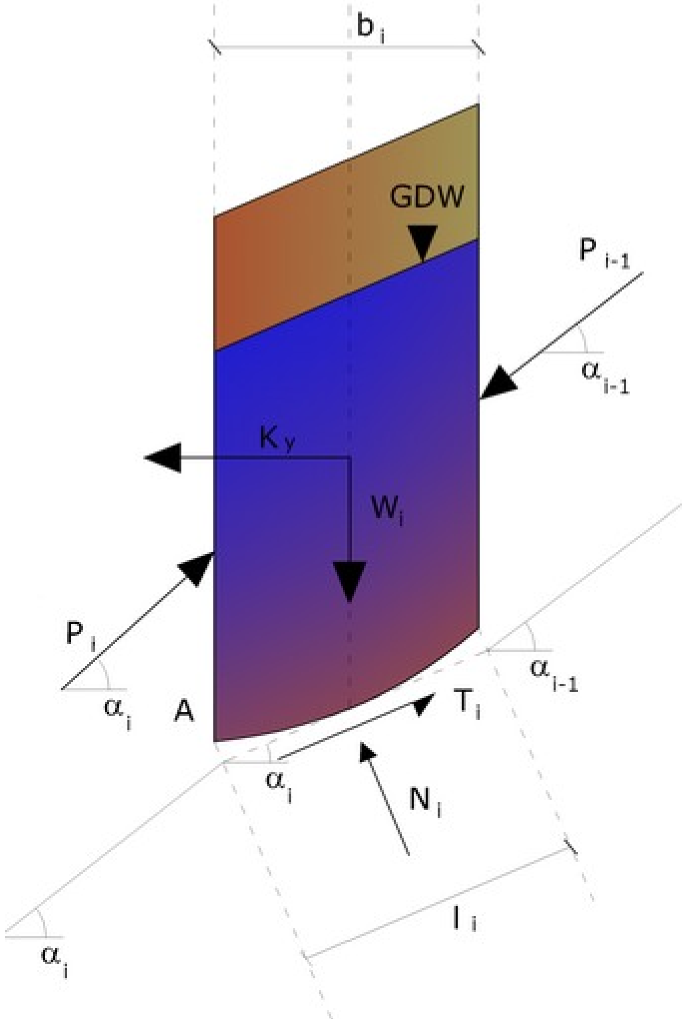
A second equation, called "*equation of moments*", is written by imposing the condition of equilibrium to rotation with respect to the middle of the base:

$$X=\frac{d\left(E_\gamma\right)}{dx}-\gamma\frac{dE}{dx}$$

these two equations are extended by integration at the whole mass concerned from sliding.
The calculation method satisfies all equilibrium equations and is applicable to surfaces of any shape, but not necessarily imply the use of a computer.

1.20.1.8 Zeng Liang method (2002)

Zeng and Liang carried out a series of parametric analyzes of a two-dimensional model developed by finite element code, which reproduces the case of drilled shafts (piles immersed in a moving soil).
The two-dimensional model reproduces a slice of soil of unit thickness and assumes that the phenomenon occurs in plane strain conditions in the direction parallel to the axis of the piles.
The model was used to investigate the influence on the formation of an arch effect of some parameters such as the distance between the piles, the diameter and the shape of the piles, and the mechanical properties of the soil. The authors identify the relation between the distance and the diameter of the piles (s/d), the dimensionless parameter determining the formation of the arch effect.
The problem appears to be statically indeterminate, with degree of indeterminacy equal to (8n-4), but in spite of this it is possible to obtain a solution by reducing the number of unknowns and thus taking simplifying assumptions, in order to make determined the problem.



Actions on the i-th slice according to Zeng Liang

The assumptions that make the problem determined are:

- Ky are taken horizontal to reduce the total number of unknowns with (n-1) to (7n-3);
- Le normal forces at the base of the strip acting in the midpoint, reducing the unknowns with n to (6n-3);
- The position of the lateral thrusts is to one-third of the average height of inter-slice and reduces the unknowns with (n-1) to (5n-2);
- The forces (P_i-1) and P_i are assumed parallel to the inclination of the base of the strip (α_i), reducing the unknowns with (n-1) to (4n-1);
- It is assumed a single yield constant for all the slices, reducing the unknowns with (n) to (3n-1);

The total number of unknowns is then reduced to (3n), to be calculated using the factor of load transfer. Furthermore, it should be noted that the stabilization force transmitted on the soil downstream of the piles is reduced by an amount R, called reduction factor, calculated as:

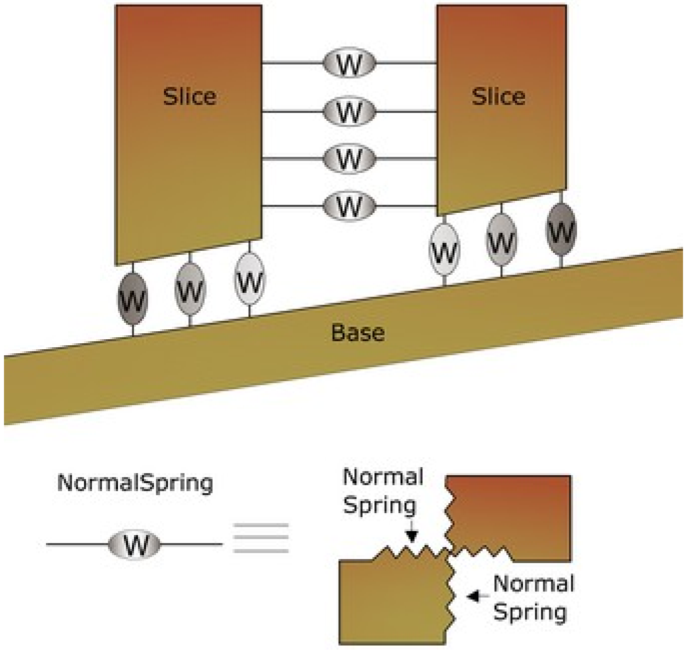
$$R = \frac{1}{s/d}\frac{1}{s/d} + \left(1 - \frac{1}{s/d}\frac{1}{s/d}\right) \cdot R_d$$

The factor R therefore depends on the ratio between the distance present between the piles and the diameter of the piles and by the factor R_d which takes into account the arch effect.

1.20.2 Numerical method

1.20.2.1 Discrete Element Method (DEM)

With this method, the soil is modeled as a series of discrete elements, which we will call "slices", and takes into account the mutual compatibility between the slices. A slope in the present model is treated as comprised of slices that are connected by Elastoplast Winkler springs. One set of springs is in the normal direction to simulate the normal stiffness. The other set is in the shear direction to simulate the sliding resistance at the interface. The behavior of the normal and shear springs is elastoplast. The normal springs do not yield in compression, but they yield in tension, with a small tensile capacity for cohesive soil (tension cutoff) and no tensile capacity for frictional soil.



Schematic Figure of Winkler Springs at Interface between Two Adjacent Slices or between Slice and Immovable Base

The shear springs yield when the shear strength is reached, and two types of behaviors are distinguished: *brittle soil* and *nonbrittle soil*. For brittle soil, the peak strength of the shear springs is determined by:

$$\tau_p = c_p + \sigma_n \cdot \tan \varphi_p$$

While the residual shear strength is given by:

$$\tau_r = c_r + \sigma_r \cdot \tan \varphi_r$$

For simplicity, in the following analysis, it is assumed that after reaching the peak strength, the soil resistance drops immediately to the residual strength value.
For plastic nonbrittle soil, the strength does not reduce at large shear deformation; thus, the residual strength has the same value as the peak strength. The formulation of the present method follows that of previous research of **Chang** and **Mistra** on the mechanics of discrete particulates.

1.20.2.2 Finite Element Method (FEM)

For the theoretical one can refer to the software GFAS (Geotechnical and F.E.M. analysis System) made by GeoStru.

2 Slope/MRE

M.R.E. (Mechanically Stabilized Earth)

Software for sizing and verification of reinforced earth, either with metallic elements or geomembranes.
It is possible to define more reinforced earth typologies in the same file and run, simultaneously, all the verification and project analysis for more load combinations.
The software allows an easy input through a series of dedicated instruments like automatic generation of the reinforcements' position, profile of reinforced earth with the option to choose a profile of constant inclination or a terraces one; the software also includes an integrated geogrid database.
The verification and project analysis can be performed during the input phase, so that it can be established which of the combinations is most unfavorable.

STANDARDS

The Standards that can be chosen for the geotechnical and structural calculation are European Union Eurocodes and also specific British, Italian and Romanian National legislation as listed below.

Eurocodes

Eurocode 2

Projects involving concrete structures. Part 1-1: General Rules.

Eurocode 7

Geotechnic Projects. Part 1: General Rules.

Eurocode 8

Project guidelines for structural resistance to seismic events. Part 5: Foundations, Retaining structures, and geotechnical aspects.

Italian National legislation:
(quoted from relevant legislation)

D.M. 11 Marzo 1988
"Norme tecniche riguardanti le indagini sui terreni e sulle rocce, la stabilità dei pendii naturali e delle scarpate, i criteri generali e le prescrizioni per la progettazione e il collaudo delle opere di sostegno delle terre e delle opere di fondazione".

D.M. 9 Gennaio 1996
"Norme tecniche per il calcolo, l'esecuzione ed il collaudo delle strutture in cemento armato, normale e precompresso e per le strutture metalliche".

D.M. 16 Gennaio 1996
"Norme tecniche per le costruzioni in zone sismiche".

STAS 3300/85; 10107/0-90 (Romanian National Standards)

BS 8006: international British Standard 8006, for analyzing ultimate limit state and serviceability limit state of the structure; both are defined by load factors and partial coefficients.

REINFORCEMENT TYPOLOGIES

Metallic strips or bars;
Geotextile strips or geotextile sheets (geomembranes)
Grids;
Reinforcement with blocks;
The program has a database of the main reinforcement elements on the market;
The database can be easily completed and modified by the user.

PROJECT

Determination of effective length and folding length, sizing of resistant section.

VERIFICATION

Pullout/Sliding;
Intern Tieback and Compound;
Global stability: Sliding, Overturning and Limit load.

VISUALIZATION

Diagram of the pressures on the work;

Diagram of the efforts in the reinforcements;

Diagram of the pressures in foundation;

Breaking wedge.

The program also offers a detailed computation report with lots of theory notes.

2.1 Internal verification

Internal verifications performed are:

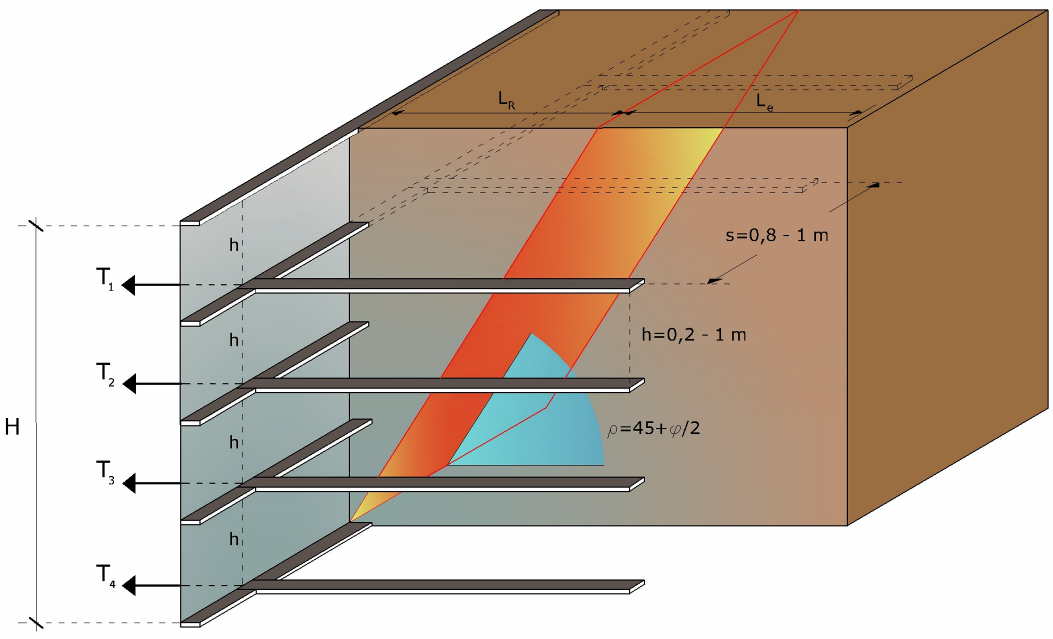
- **Pullout**
- **Sliding**
- **Tensile strength**

For both pullout and sliding are sized effective lengths such as to develop friction forces that contrast the tensile force induced in the reinforcements, this is performed ensuring a safety factor previously assigned.

The verification of the tensile strength consists in sizing the section in the reinforcement in order to obtain induced stresses inferior to the admissible one.

2.1.1 Reinforcenets spacing

Vertical and horizontal strip spacing in the reinforcement is calculated like in figure above.
This does not apply if the reinforcement is realized with metallic nets or with geotextile sheets, that have fixed horizontal spacing. The vertical spacing can vary approximately from 0.2 to 1 m while the horizontal one it is approximately between 0.8 and 1 m.
The analysis refers to a unit width segment, to which is associated the horizontal pressure diagram.



Schematic representation of the arrangement of reinforcements and relative spacing

2.1.2 Reinforcements' tensile forces

Tensile forces in various reinforcements, given by the diagram area of the pressures related to every strip, are determinate.
For the triangular diagram related to the embankment, the force in the strip is given by the area of the trapezium element *ab'd'* and it is transformed in average pressure q_i at Z_i strip depth using the relation:

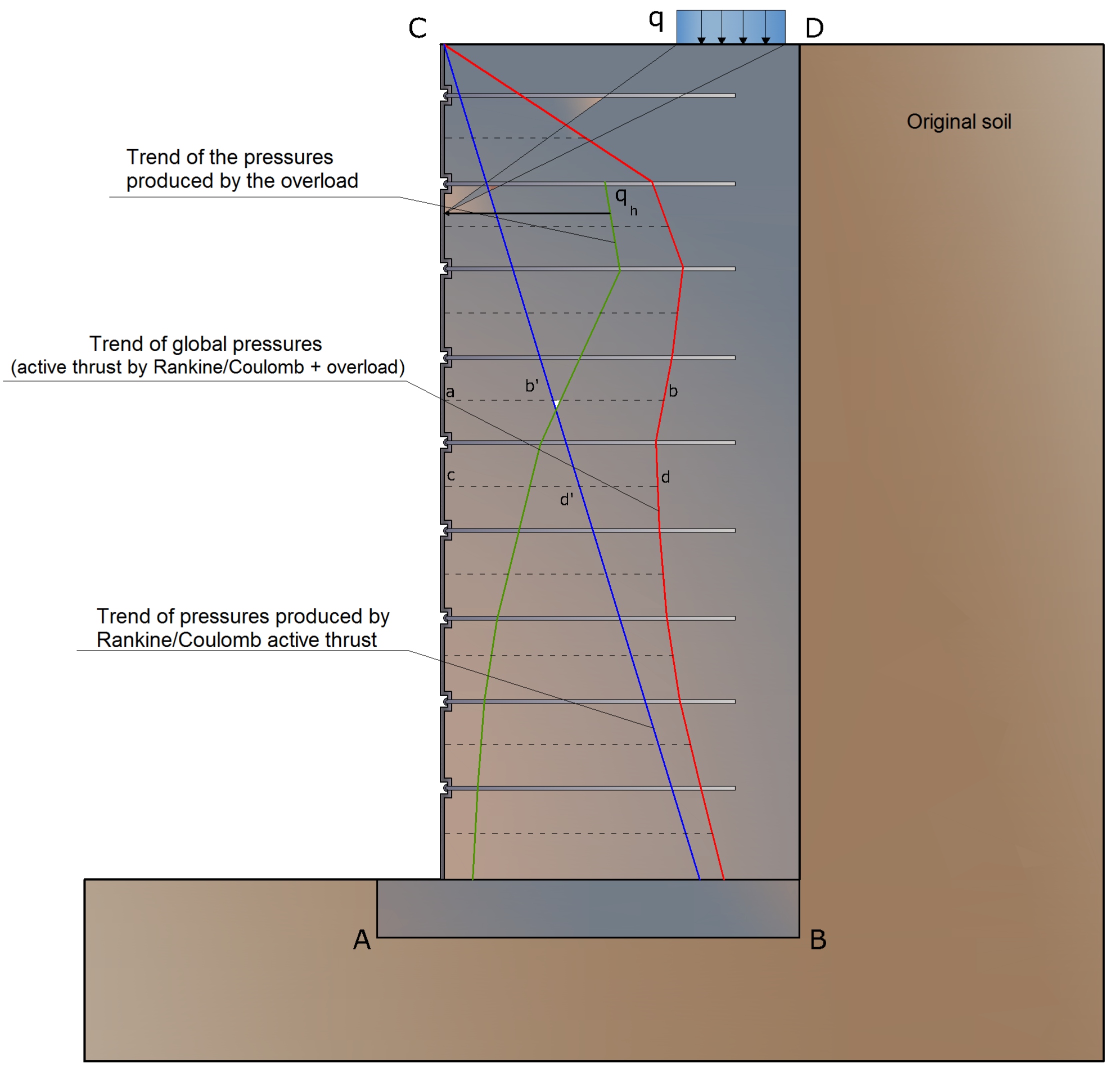
$$q_i = \gamma \cdot z_i \cdot K_a$$

The q_i pressure acts on an area defined by the hxs armature spacing and corresponds to a tensile force in the reinforcement equal to:

$$T_i = q_i \cdot A = \gamma \cdot z_i \cdot K_a \cdot (h \cdot s)$$

For equilibrium the sum of the tensile forces must be equal to the horizontal component of the acting forces.

$$\Sigma\ T_i = P_{ah}$$



Schematic representation of the arrangement of reinforcements and their spacing

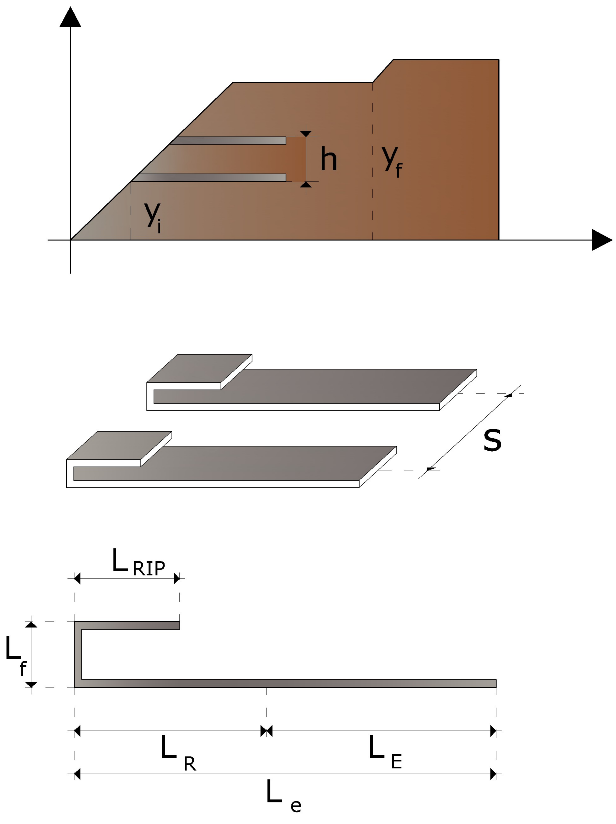
2.1.3 Effective lengths

The strip lengths Le that appear in figure below will be determined so that the reinforcements will develop a frictional force that satisfies the equality $F_r = T_i$

On the base of these lengths and of the Rankine's wedge dimensions, the global length Lo of the strips to use can be determined. Generally, the strips used have the same length for all the height of the wall.

The anchorage length depends on the friction coefficient $f = \tan(\delta)$ between soil and reinforcement, being d an opportune fraction of the internal friction angle of the soil f.

If the strip is rough enough $\delta=\varphi$, while for smooth metals d is about 20° to 25°.

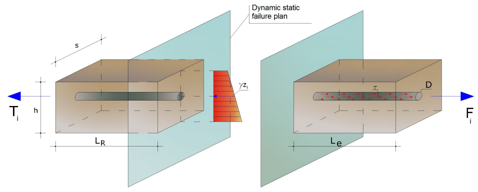


Schematic representation of reinforcement length

For strips of $b \cdot L_e$size or for unitary width and Le, length geotextile sheets, both faces develop friction; for circular bars, the frictional resistance is developed along the perimeter. In either case, the friction is given by the product of f and the normal pressure at the reinforcement calculated as $p_0 = \gamma \cdot z_i$ where z_i is the average distance from the soil surface to the reinforcement.

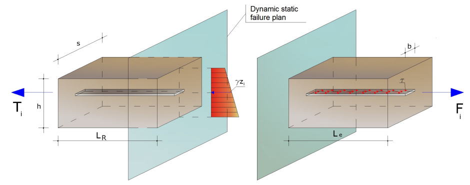
For bars

$$F_i = \pi D(\gamma z_i)\tan\delta\,(L_e) \geq T_i$$



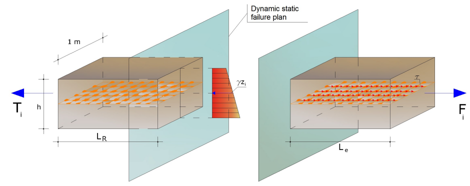
For strips

$$F_i = 2(\gamma z_i)\tan\delta\,(b \times L_e) \geq T_i$$



For sheets

$$F_i = 2(\gamma z_i)\tan\delta\,(1 \times L_e) \geq T_i$$



If in the previous formulas the sign ≥ was replaced with the equal sign, the safety factor Fs is equal to 1. If is assumed Fs > 1, the L_e value is necessarily bigger than the one given by these formulas.

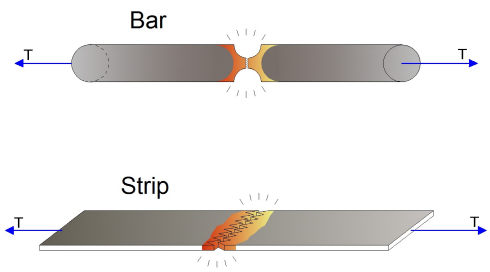
2.1.4 Tensile strength

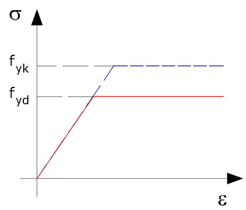
Once the tensile forces in the reinforcements are known T_i, the armatures section b · t is determined. For metallic bars or strips with allowable stress equal with:

$$f_a = \frac{f_y}{FS}$$

we have:

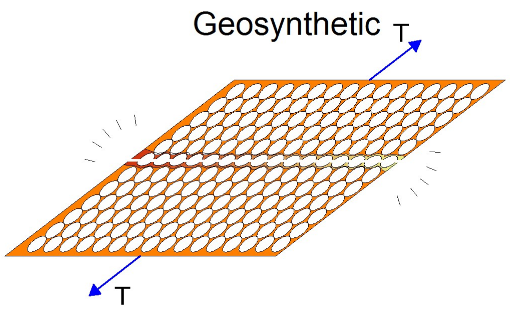
$$\frac{\pi \cdot D^2}{4} \cdot f_a \geq T_i$$

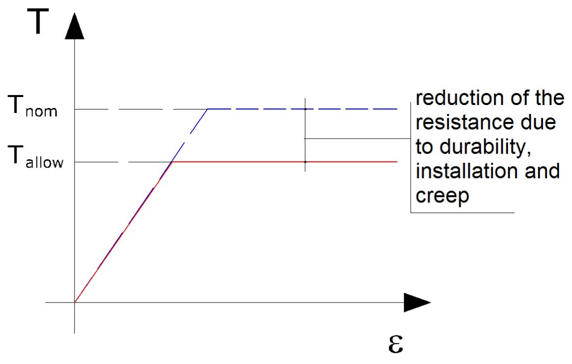




For geotextiles a problem that occurs in that the fabrics resistance varies from one producer to another. From available ones select one where:

(Strip length b) $\cdot$ *(resistance per width unit)* $\geq T_i$





The allowable resistance used for sizing is:

$$\frac{LTDS}{FS}$$

The unique safety factor (FS) is the combination of: Fs_{joint}, $Fs_{chemical}$, $Fs_{biological}$, $Fs_{environmental\ damages}$

$$FS = Fs_{joint} \cdot Fs_{chemical} \cdot Fs_{biological} \cdot Fs_{environmental\ damages}$$

2.1.5 Folding length

The calculation of the length of the folding is conducted in such manner to prevent the bulging of the facade. This length, however, has as a minimum value of one meter.

2.1.6 Tieback & Compound

Besides sliding and pullout internal verification it is possible to carry out the verification of the containment works in regard to potential failure surfaces.

The verifications are: Tieback & Compound

Tieback analysis (internal stability analysis)

This kind of verification is useful to determine if the tensile strength of each reinforcement is sufficient to ensure the reinforced earth from possible internal collapses due to its own weight and to overloads. It therefore ensures against any sliding along surfaces that emerge on the face of reinforced earth. The ability of tensile strength of the reinforcement is calculated in order to determine whether the anchorage of the reinforcement in the ground is such as to avoid the collapse for sliding along the potential failure surface. Tieback analysis is performed with the classical methods of slope stability like bound computation by one point point corresponding to the position of each reinforcement on the face of the work. This analysis allows obtaining an even distribution of the stresses in the reinforcements.

Compound analysis (composed stability analysis)
The use of Tieback analysis ensures against any internal damages; on the other hand the capacity of the reinforcements to develop their own resistance depends on their pull-out resistance and, therefore, on their anchoring in a stable zone. However for deeper sliding surfaces and/or sliding surfaces passing through the foot of the slope, these resistances can be reduced and therefore cause instability. Therefore it becomes necessary a stability analysis that allows to determine whether the length of the first k reinforcements is such as not to cause sliding along the sliding surfaces above. This is definitely a conservative verification but it ensures the translational and rotational stability of the whole complex.

The analysis is carried out with the limit equilibrium methods, for both circular and free form surfaces.

2.2 Global verification

The stability of the work is checked as a whole considering it as a rigid body. The work is secure when when is verified the safety to:

- *Overturning*
- *Sliding*
- *Limit load*
- *Global stability*

Overturning verification
The overturning is represented by the possible rotation of the work with respect to the downstream point.

The action that determines the overturning is given by the horizontal component of the earth's thrust plus eventual external actions.
The stabilizing action is given by the vertical component of earth's thrust, by work's own weight.
The stabilizing action represented by the action of the earth's passive thrust is not taken into consideration.
In analytical terms the overturning verification can be expressed with the condition that the stabilizing moment Ms, with respect to the rotation center, is non inferior to the moment induced by the overturning forces Mr, with respect to the rotation center.
The security of this equilibrium must be ensured with a due safety coefficient.

Sliding verification

The sliding depends on the possibility that the parallel forces to the contact plan between foundation and soil are superior to the soil-foundation friction forces.
The force that determines the sliding T is the horizontal component of the thrust plus eventual surcharges, while the force that opposes the sliding is given by the resultant of the normal forces N to the contact plan multiplied by the friction coefficient.
The friction coefficient f is the tangent of the foundation-soil friction angle.

To reduce sliding danger the foundation footing can be inclined.

In analytical terms it can be expressed as:

$$N \cdot f > FS \cdot T$$

FS, safety factor, varies with the standard.

Limit load verification

It is carried out comparing the maximum normal stress on the foundation's footing with the ultimate shear stress of the soil.
This condition is considered verified if the ratio between the limit stress and the maximum stress is superior to a fixed safety factor.

Global stability verification

Consists in checking the rotation of a soil cylinder containing both the work and the thrust wedge. The verification is performed by software Slope made by GeoStru.

2.2.1 Thrust

Active thrust

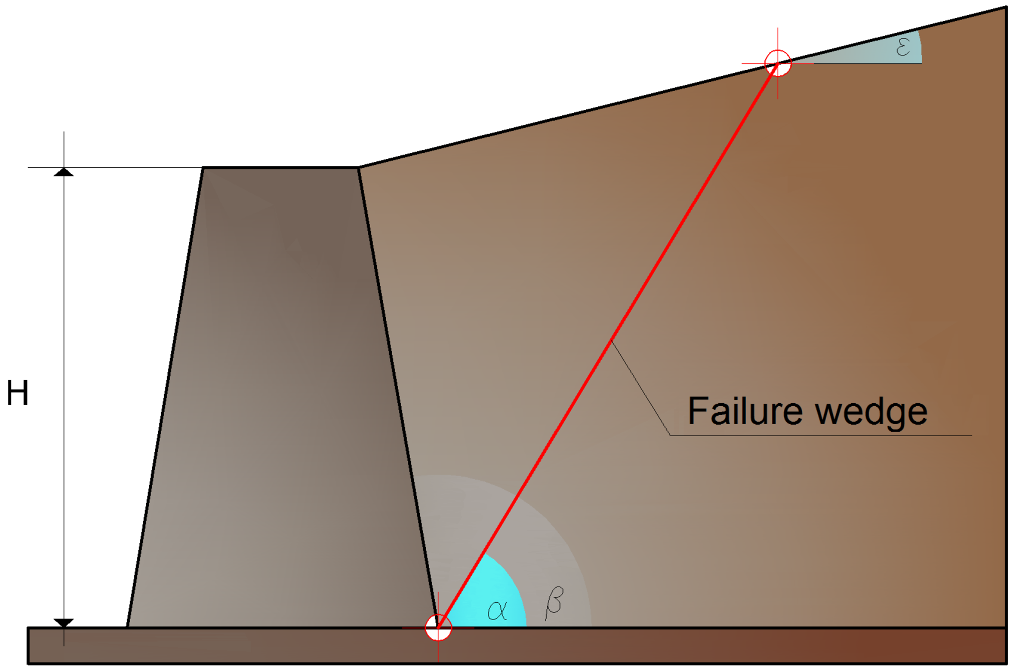
Active pressure calculation using Coulomb's method is based on global limit equilibrium theory of a system whose components are the wall and the wedge of homogeneous terrain behind the work assuming rough surface.

Where terrain is dry and homogeneous the pressure diagram is expressed linearly by the following:

$$P_t = K_A \cdot \gamma_t \cdot z$$

Thrust S_t is applied at 1/3 H with the value:

$$S_t = \frac{1}{2}\gamma \cdot H^2 \cdot K_A$$



Representation of the failure wedge on the back of the wall

Values of K_a

$\delta < (\beta - \phi - \varepsilon)$ according to Muller-Breslau

Where:

γ_t	Soil unit weight;
β	Inclination of the inner wall with respect to the horizontal plane passing through the footing;
φ	Soil angle of shearing resistance;
δ	Soil-wall friction angle;
ε	Inclination of the ground surface from the horizontal, positive if counterclockwise;
H	Wall height.

Active pressure calculation according to Rankine

If $\varepsilon = \delta = 0$ and $\beta = 90°$ (wall with smooth surface and backfill with horizontal surface) thrust S_t is simplified to:

$$S_t = \frac{\gamma \cdot H^2 (1 - \sin\varphi)}{2(1 + \sin\varphi)} = \frac{\gamma \cdot H^2}{2} tg\left(45 - \frac{\varphi}{2}\right)$$

that coincides with Rankine's equation for gives active pressure computation where backfill is horizontal.

Effectively Rankine used the same hypothesis as Coulomb except that he ignored wall-soil friction and cohesion. Rankine's expression for K_A in general form is as follows:

$$K_A = \cos\varepsilon \frac{\cos\varepsilon - \sqrt{\cos^2\varepsilon - \cos^2\varphi}}{\cos\varepsilon + \sqrt{\cos^2\varepsilon - \cos^2\varphi}}$$

Active pressure calculation according to Mononobe & Okabe

Mononobe & Okabe's evaluation of active thrust concerns thrust in seismic states with pseudo-static method. This is based on global limit equilibrium theory of a system whose components are the wall and the wedge of homogeneous terrain behind the work and involved in the failure in an artificial computation configuration in which ε, field level inclination angle with respect to the horizontal, and the angle β -

inclination of internal wall surface to the horizontal -are increased by an amount θ such that:

$$tg\theta = \frac{k_h}{(1 \pm k_{v)}}$$

where k_h is the horizontal seismic coefficient and k_v is the vertical seismic coefficient.

Where no specific studies exist, coefficients k_h an k_v should be calculated as:

$$k_h = \frac{S \cdot a_g}{r} \qquad k_v = 0{,}5 \cdot k_h$$

where $S \cdot a_g$ is the maximum seismic acceleration in the various categories of the stratigraphic profile.

Factor r can take the value r=2 where the work is one of some flexibility (eg. gravity walls). In all other cases (stiff reinforced concrete walls, reinforced concrete walls on piles or with anchors, fixed head walls - basement walls) it should be set to r=1.

Effect due to cohesion

Cohesion introduces negative constant pressures equal to:

$$P_c = -2 \cdot c \cdot \sqrt{K_A}$$

As it is not possible to calculate a priori the thrust reduction induced in the thrust by cohesion a critical height Z_c has been calculated as:

$$Z_c = \frac{2 \cdot c}{\gamma} \cdot \frac{1}{\sqrt{K_a}} - \frac{Q \cdot \dfrac{\sin\beta}{\sin(\beta + \varepsilon)}}{\gamma}$$

where

Q = Load acting on the backfill.

If $Z_c < 0$ the effects may be applied directly as a decrease whose value is:

applied at H/2.

Uniform load on embankment

A load Q, uniformly distributed on the ground surface generates constant pressures as:

$$P_q = \frac{K_A \cdot Q \cdot sen\beta}{sen(\beta + \varepsilon)}$$

Integrating, a thrust S_q:

$$S_q = K_A \cdot Q \cdot H \frac{\sin\beta}{\sin(\beta + \varepsilon)}$$

Applies at H/2, indicating with K_A the active thrust coefficient according to Muller-Breslau.

Active thrust in seismic state

In seismic state the calculation force exercised by the embankment on the wall is given by:

$$E_d = \frac{1}{2}\gamma(1 \pm k_v) \cdot K \cdot H^2 + E_{ws} + E_{wd}$$

where:

H = wall height

k_v = vertical seismic coefficient

γ = unit weight

K = coefficients of total active thrust (static + dynamic)

E_{ws} = hydrostatic thrust of water

E_{wd} = hydrodynamic thrust

For impermeable soils, hydrodynamic thrust E_{wd} = 0, but a correction on evaluation of the angle q in Mononobe & Okabe's formula is made as follows:

In highly permeable soils in seismic states, the same correction is applied but hydrodynamic thrust assumes the following value:

$$E_{wd} = \frac{7}{12} k_h \gamma_w H'^2$$

where H' is the height of the groundwater table (Gwt) from the base of the wall.

Hydrostatic thrust

Gwt whose surface is at height H_w from the base of the wall generates hydrostatic pressures normal to its surface that, at depth z, are expressed as:

$$P_w(z) = \gamma_w \cdot z$$

With a resultant equal to:

$$S_w = 1/2 \cdot \gamma_w \cdot H^2$$

The thrust of the submerged terrain can be obtained substituting γ_t with γ'_t (γ'_t = $\gamma_{saturated}$ - γ_w) , effective weight of submerged material.

Passive resistance

In homogeneous soil a linear diagram of pressures results:

$$P_t = K_p \cdot \gamma_t \cdot z$$

integrating is obtained the passive thrust:

$$S_p = \frac{1}{2} \cdot \gamma \cdot H^2 \cdot K_p$$

having:

$$K_p = \frac{\sin^2(\varphi + \beta)}{\sin^2 \beta \cdot \sin(\beta - \delta) \cdot \left[1 - \sqrt{\frac{\sin(\delta + \varphi) \cdot \sin(\varphi + \varepsilon)}{\sin(\beta - \delta) \cdot \sin(\beta - \varepsilon)}}\right]^2}$$

(Muller-Breslau) with d limit values:

$$\delta < \beta - \varphi - \varepsilon$$

The expression of K_p according to Rankine assumes the following form:

$$K_p = \frac{\cos\varepsilon + \sqrt{\cos^2\varepsilon - \cos^2\varphi}}{\cos\varepsilon - \sqrt{\cos^2\varepsilon - \cos^2\varphi}}$$

2.2.2 Limit load

Brinch - Hansen (EC 7 – EC 8)

In order for the foundation of a wall to safely sustain the design load in regard to general failure, the following inequality must be satisfied:

$$V_d \le R_d$$

where:

V_d is the design load, normal to the footing, including the weight of the wall;

R_d is the design limit load of the foundation for normal loads, also taking into account eccentric and inclined loads.

In the analytical evaluation of the design limit load R_d must be considered situations for short and long term in fine-grained soils. The design limit load in **undrained conditions** is calculated as:

$$\frac{R}{A'} = (2+\pi)\cdot c_u \cdot s_c \cdot i_c + q$$

where:

$A' = B' \cdot L'$	Design effective foundation area. When eccentric loads are involved, use the reduced area with the resultant load applied at the center for the area.
c_u	Undrained cohesion
q	Total lithostatic pressure to the substrate
s_c	Form factor
	Form factor for rectangular foundations
	Form factor for square or circular foundations.

$$i_c = 0.5 \cdot \left(1 + \sqrt{1 - \frac{\quad}{A}} \right.$$

Correction factor for the inclination of the load due to a load H.

Design limit load in **drained conditions** is calculated as follows:

$$\frac{R}{A'} = c' \cdot N_c \cdot s_c \cdot i_c + q' \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot \gamma' \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma$$

where:

$$N_q = e^{\pi \tan\varphi'} \tan^2\left(45 + \frac{\varphi'}{2}\right)$$

$$N_c = (N_q - 1)\cot\varphi'$$

$$N_\gamma = 2(N_q + 1)\tan\varphi'$$

represent the bearing capacity factors and are functions of the internal friction angle. The form factors are defined by the following relations and were determined in part analytically in part by empirical procedures.

$$s_q = 1 + sen\varphi'$$ for a square or circular shape

$$s_\gamma = 1 - 0.3 \cdot \left(\frac{B'}{L'}\right)$$ for rectangular shape

$$s_\gamma = 0.7$$ for square or circular shape

$$s_c = \frac{(s_q \cdot N_q - 1)}{(N_q - 1)}$$ for rectangular, square or circular shape

$$s_q = 1 + \left(\frac{B'}{L'}\right) sen\varphi'$$ for rectangular shape

Resultant inclination factors due to a horizontal load H parallel to B'

$$i_q = \left[1 - \frac{0.7H}{\left(V + A^{'} \cdot c^{'} \cdot \cot\varphi^{'}\right)}\right]^3$$

$$i_\gamma = \left[1 - \frac{H}{\left(V + A^{'} \cdot c^{'} \cdot \cot\varphi^{'}\right)}\right]^3$$

$$i_c = \frac{\left(i_q \cdot N_q - 1\right)}{\left(N_q - 1\right)}$$

If H forms an angle θ with the direction of L′ :

$$i_q = i_\gamma = 1 - \frac{H}{\left(V + A^{'} \cdot c^{'} \cdot \cot\varphi^{'}\right)}$$

$$i_c = \frac{\left(i_q \cdot N_q - 1\right)}{\left(N_q - 1\right)}$$

In addition to the correction factors mentioned above are also considered the ones complementary to the depth of the bearing surface and of the ground level (Hansen).

2.3 General data

Code

Name of typology: needed for identification.

Description

Work's description.

Reinforcements list

Reinforcement typology archive is a database of materials that can be personalized by the user, just select Reinforcement List and press the mouse's right button to add or to delete a reinforcement.

The data required varies as a function of the type: bar, strip or sheet.

For each of those, beside the geometrical identifiers, it is necessary to assign the material's allowable resistance fa.

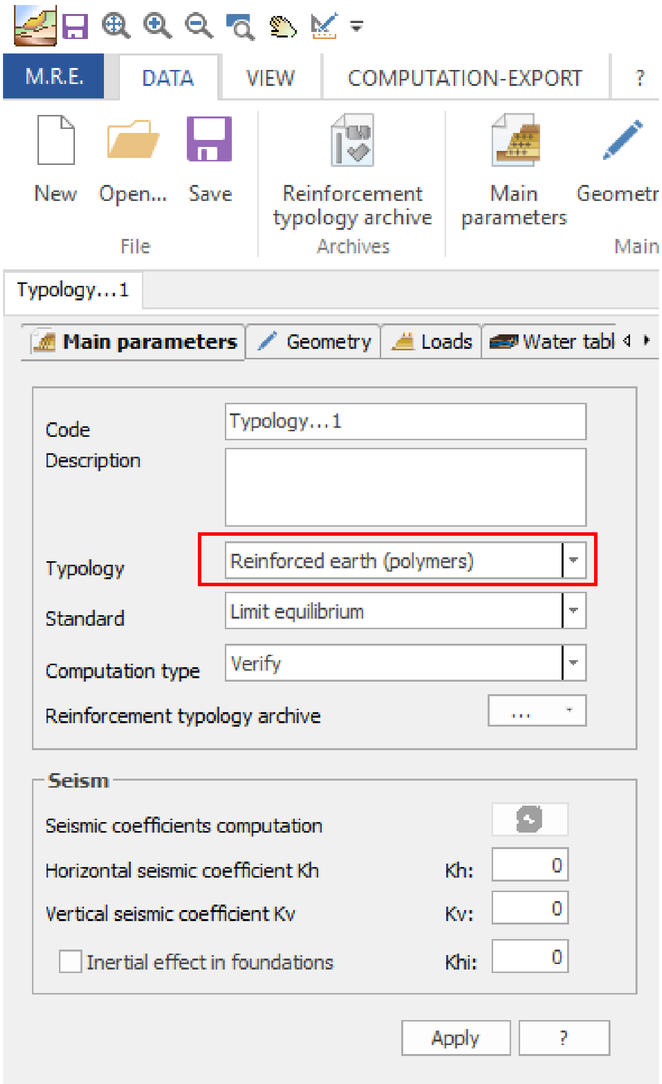
Design standard

The user can choose one of the following standards:

Limit equilibrium: applies the limit equilibrium theory with just one combination of load and a global safety factor on the various verifications.
Norme Tecniche *(Testo Unico)*: si possono considerare più combinazioni di carico con differenti fattori di combinazione e i coefficienti di sicurezza parziali sui parametri geotecnici.
BS 8006: international British Standard 8006, for analyzing ultimate limit state and serviceability limit state of the structure; both are defined by load factors and partial coefficients.
EC8: the Eurocode provide for ultimate limit state and serviceability limit state analysis.
Depending on the chosen Standard the program generates the load combination to analyze. It is possible to choose to perform the computation with the designing or verification criterion.

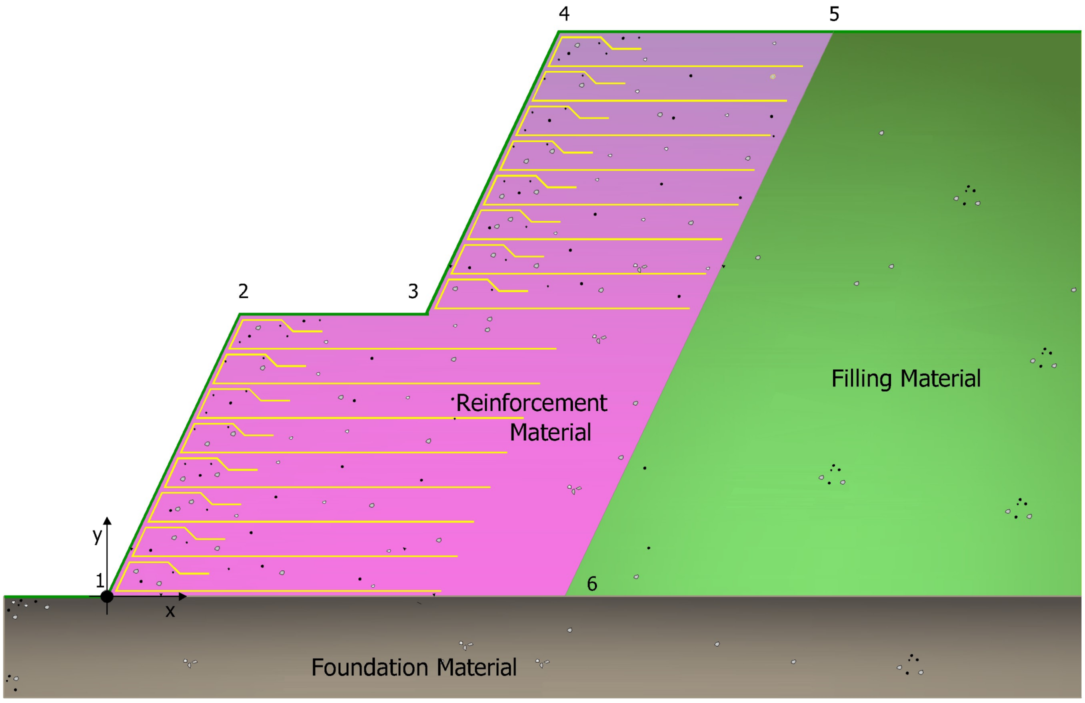
Seism
The program calculates the horizontal and vertical thrust coefficient on the base of the maximum acceleration on soil. The user may modify these coefficients.

Typology
After choosing the typology from general data and confirmed by pressing apply, the software will select the specific theoretical part to be introduced in the final report that can be generated from the menu Report - Export in Word.



2.4 Geometrical data

The reinforced earth profile is defined by X, Y coordinates referred to a local reference system whose origin is the lower vertex. The profile should be closed as seen in the figure below, the vertices of the reinforced earth *1,2,3,4,5,6* close the reinforcement polygon shown in pink.



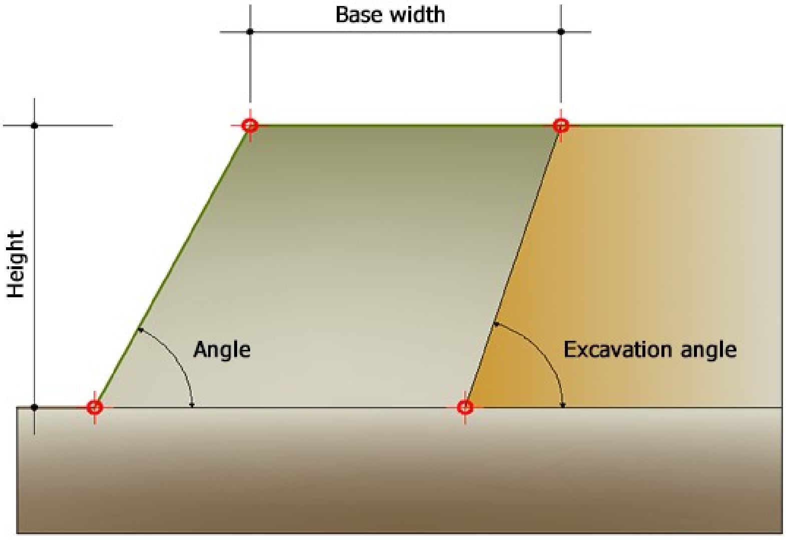
Note:

The polygon of the reinforcements must be closed the first vertex must have coordinates (0,0).

The polygon may be defined by points, inserting manually the coordinates X, Y in the table, or by automatic generation, in which case the geometric profile can be either a constant slope or a terraced slope.

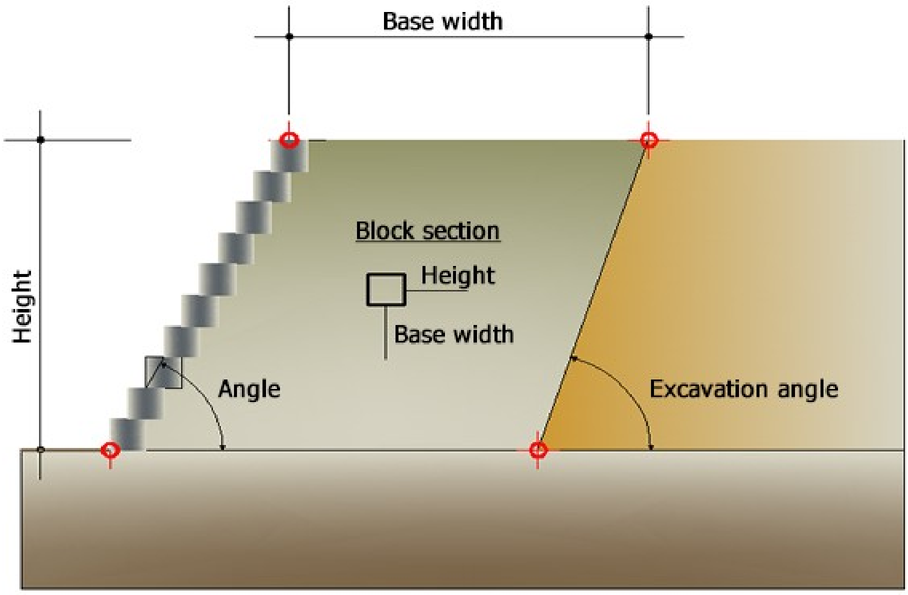
If automatic generation is chosen, the required input data is:

- the reinforcement angle, internal to the polygon, the angle that the profile of the slope forms with the horizontal
- the height and the width of the reinforcement polygon



Geometry automatic generation, input data

The automatic generation mode "Terracing" is performed by the software giving the size of the blocks by default or by using those that are assigned by the user in "Blocks" section.



In "Geometry" are assigned the geometrical dimensions of the foundation. Confirm each input with "Apply".
To delete the input select all numerical values in the table and press Del key from your keyboard, or right click and choose "Delete all".
The command "Import DXF" offers the possibility to import the profile of the reinforcement, a necessary condition. The profile is defined by an open polyline increasing from left to right.

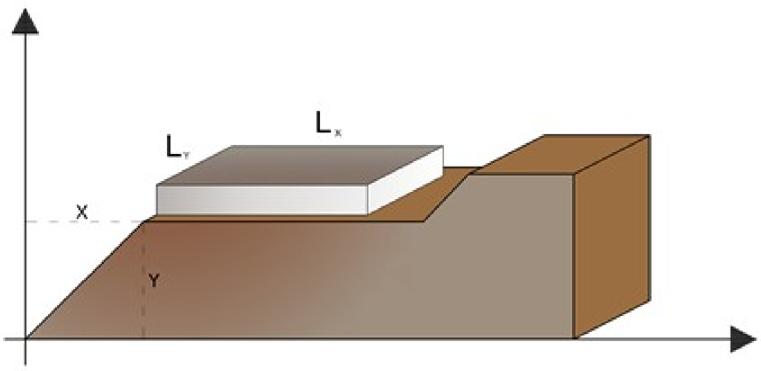
If the user has the profile coordinates of a spreadsheet they can be copied and pasted in the input table.

The meaning of the flags in the "Use" column is that the corresponding point has been used to "hook" a block.

2.5 Loads

In the section "*Loads*" can be defined the loads that act on the reinforced earth and are identified by the following parameters:

Description: insert a name to the surcharge - it will be shown in the "*Perform analysis*" table from where the user can choose if the surcharge is taken into consideration or not in the current combination.
Input point X and Y: regarding the local system in which reinforced earth was defined.
Lengths L_x, L_y: are defining the load width.
Q: surcharge



Definition of geometric parameters of the load

2.6 Position of the reinforcements

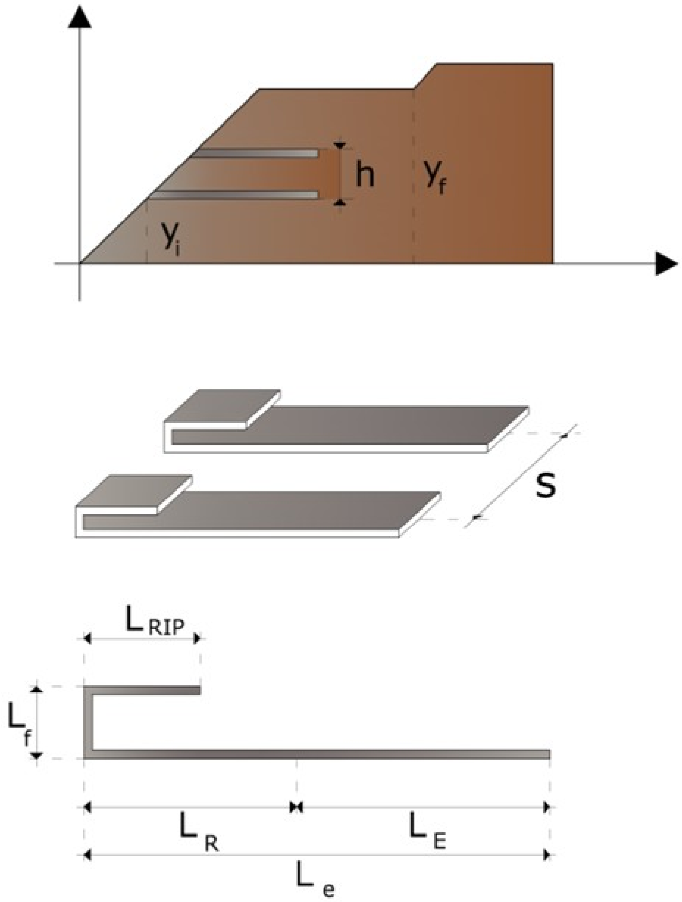
The position of the reinforcements can be defined by points in the table by entering the coordinates X, Y with respect to the Cartesian reference system represented in the figure below.

To facilitate the reinforcements positioning it is possible to proceed with the automatic generation assigning the following geometrical characteristics:

- Initial position Y_i
- Final position Y_f
- Spacing *h*
- Bending length L_{RIP}
- Front length L_f
- Total length L_t

It is possible to insert the distance between the reinforcements and their type from Reinforcement typology archive.
After the input a double command must be performed:
- "*Generate*" calculates the insertion position of the reinforcements represented by a colored dot
- "*Apply*" makes the dot disappear and shows the graphical representation of the chosen reinforcements instead.

The table will contain all the variables useful for the calculation or verification of reinforcements, the user may at any time modify confirming the change with the command "*Apply*".



Definition of geometric parameters of the reinforcements

The analysis is performed always for one linear meter of thickness.

2.7 Safety factors

The safety factors must be set with reference to the selected standard. The software divides them in two categories: *global verifications and composed verifications.*

Global verifications

- *Sliding*: the verification is the verification is carried out between the foundation and the first reinforcement (base)
- *Overturning*: the overturning verification is performed with respect to the left vertex of the soil base

- *Limit load*: limit load is calculated on the foundation soil and takes into account the depth of the bearing surface and the inclination (Geometrical data)

Internal global stability: are internal stability checks using the limit equilibrium methods to calculate the potential internal slip/sliding surfaces (Tieback and Compound).

2.8 Analysis

In the analysis window are displayed the load combinations to verify. Selecting Load combinations and clicking on the right mouse button you can add, delete or regenerate the list of combinations. For each combination it is possible to choose the combination coefficient for the actions, for the geotechnical parameters of the soil and resistances.
Using the command "*Computation*" are analyzed all combinations created and, for each of them, will be shown the lengths of the reinforcements inside the breaking wedge "L_R" and the effective lengths "L_E".

2.9 Results

The results are organized in tables, the table "*Blocks*" is activated if in the "*Geometry*" section was generated a terraced profile and disposed the insertion of blocks.
The first table shows for each reinforcement the results of the analysis organized as follows:

- the lengths of the facade, the folding lengths, the lengths internal to the wedge, the effective lengths and total lengths of the reinforcement
- pull-out safety factor
- stress in the reinforcement
- failure safety factor
- thrust on the reinforcement
- pull-out resistance
- ultimate strength of the material

Global safety factors for sliding, overturning and limit load of the reinforced earth are shown.

The table "*Blocks*" is structured as follows:

- positioning of the single block (Cartesian coordinates with respect to the global reference system)
- thrust of the soil on the block
- resistance of the geogrid between block and block
- overturning and sliding safety factor of the block (local checks)

The results of the global verifications are summarized by the values that the global safety factors to sliding, overturning and limit load assume.

When the computation is completed, the software offers in the output file the following diagrams:

- thrust on the reinforcement
- safety factor on the reinforcement
- failure safety factor
- pull-out resistance
- ultimate strength of the material

In "*Report*" menu, the command "*Export in Word*" allows the generation of a detailed computation report, in which are shown, in addition to theoretical notes, the results of the analysis in a tabular form.
Selecting the "*Close*" button, the verification project of the reinforced earth is closed and the user is brought in the main window of Slope to run the stability verifications Tieback and Compound. In the "*Computation*" menu one must select "*Internal verification*" and the Tieback and Compound analysis starts. Choosing the command "Internal verifications/Tieback results" the software generates a .doc file in which, at the level of each reinforcement and for the bound points are shown the critical surfaces, the safety factor and the verification type: Tieback or Compound.

3 SLOPE/ROCK

For rock slopes, different from those in soil, the Mohr-Coulomb failure criterion can not be used to define the resistance of the material; however with this method is described a procedure that allows the application of the classical methods of Limit Equilibrium even in rocky slopes.

3.1 Hoek & Bray

For rock slopes, different from those in soil, the Mohr-Coulomb failure criterion can not be used to define the resistance of the material; however with this method is described a procedure that allows the application of the classical methods of Limit Equilibrium even in rocky slopes.
In this purpose are defined the angle of shearing and the cohesion along the sliding surface according to the following expressions:

$$\mathrm{tg}\,\varphi = AB\left(\frac{N}{\sigma_c} - T\right)^{B-1}$$

$$c = A\sigma_c\left(\frac{N}{\sigma_c} - T\right)^{B} - N\mathrm{tg}\,\varphi$$

where:

σ_c is the uniaxial compressive strength of the rock;

A, B, T constants function of the lithotype and the quality of the rock (table below);

N normal stress at the base of the slice.

The constants A, B and T are determined as a function of the classification of the rocks according to Bieniawski (RMR index) or according to Barton (Q index).
Between the two classifications, on the base of 111 examples analyzed, was found the following correlation:

Lithotype Rock quality	Limestones Dolomites Marl	Shales Siltstones	Arenites Quartzites	Andesites Basalts Rhyolites	Amphibolites Gneiss Granites Gabbros
RMR =100 Q = 500	A = 0.816 B = 0.658 T = -0.140	A = 0.918 B = 0.677 T = -0.099	A = 1.044 B = 0.692 T = -0.067	A = 1.086 B = 0.696 T = -0.059	A = 1.220 B = 0.705 T = -0.040
RMR = 85 Q = 100	A = 0.651 B = 0.679 T = -0.028	A = 0.739 B = 0.692 T = -0.020	A = 0.848 B = 0.702 T = -0.013	A = 0.883 B = 0.705 T = -0.012	A = 0.998 B = 0.712 T = -0.008
RMR = 65 Q = 10	A = 0.369 B = 0.669 T = -0.006	A = 0.427 B = 0.683 T = -0.004	A = 0.501 B = 0.695 T = -0.003	A = 0.525 B = 0.698 T = -0.002	A = 0.603 B = 0.707 T = -0.002
RMR = 44 Q = 1	A = 0.198 B = 0.662 T = -0.0007	A = 0.234 B = 0.675 T = -0.0005	A = 0.280 B = 0.688 T = -0.0003	A = 0.295 B = 0.691 T = -0.003	A = 0.346 B = 0.700 T = -0.0002
RMR = 3 Q = 0.1	A = 0.115 B = 0.646 T = -0.0002	A = 0.129 B = 0.655 T = -0.0002	A = 0.162 B = 0.672 T = -0.0001	A = 0.172 B = 0.676 T = -0.0001	A = 0.203 B = 0.686 T = -0.0001
RMR = 3 Q = 0.01	A = 0.042 B = 0.534 T = 0	A = 0.050 B = 0.539 T = 0	A = 0.061 B = 0.546 T = 0	A = 0.065 B = 0.548 T = 0	A = 0.078 B = 0.556 T = 0

Relation between the classification of the rocks and the parameters A, B and T

4 SLOPE/D.E.M.

With this method, the soil is modeled as a series of discrete elements, which we will call "slices", and takes into account the mutual compatibility between the slices. A slope in the present model is treated as comprised of slices that are connected by elastoplastic Winkler springs. One set of springs is in the normal direction to simulate the normal stiffness. The other set is in the shear direction to simulate the sliding resistance at the interface. The behavior of the normal and shear springs is elastoplastic. The normal springs do not yield in compression, but they yield in tension, with a small tensile capacity for cohesive soil (tension cutoff) and no tensile capacity for frictional soil.

4.1 D.E.M.

Interfacing between the slices

With this method, the soil is modeled as a series of discrete elements, which we will call "slices", and takes into account the mutual compatibility between the slices. A slope in the present model is treated as comprised of slices that are connected by elastoplastic Winkler springs. One set of springs is in the normal direction to simulate the normal stiffness. The other set is in the shear direction to simulate the sliding resistance at the interface. The behavior of the normal and shear springs is elastoplastic. The normal springs do not yield in compression, but they yield in tension, with a small tensile capacity for cohesive soil (tension cutoff) and no tensile capacity for frictional soil.

The shear springs yield when the shear strength is reached. For brittle soil, the peak strength of the shear springs is determined by:

while the residual shear strength given by:

For simplicity, in the following analysis, it is assumed that after reaching the peak strength, the soil resistance drops immediately to the residual strength value.

For plastic non brittle soil, the strength does not reduce at large shear deformation; thus, the residual strength has the same value as the peak strength. The formulation of the present method follows that of previous research of Chang and Mistra on the mechanics of discrete particulates.

Let u_i^a, u_i^b, e ω^a, ω^b represent the translations and rotations of slice A and slice B, respectively. Let P be the midpoint of the interface between these slices. The displacement of slice B relative to slice A, at point P, is then expressed in the figure below, where riap the vector joining the centroid of the slice to location P. The displacement of the slice A in the point P is expressed as follows:

$$\begin{Bmatrix} \Delta_x^p \\ \Delta_y^p \\ \Delta_\omega^p \end{Bmatrix} = \begin{bmatrix} 1 & 0 & -r_y^{bp} \\ 0 & 1 & r_x^{bp} \\ 0 & 0 & 1 \end{bmatrix} \cdot \begin{Bmatrix} u_x^b \\ u_y^b \\ \omega^b \end{Bmatrix} - \begin{bmatrix} 1 & 0 & -r_y^{ap} \\ 0 & 1 & r_x^{ap} \\ 0 & 0 & 1 \end{bmatrix} \cdot \begin{Bmatrix} u_x^a \\ u_y^a \\ \omega^a \end{Bmatrix} \qquad (3)$$

If slice B is immovable, the values of u_x^b, u_y^b, e ω^b taken equal to zero .

Let nip be an inward vector normal to the side face of slice A at point P, defined as $n_i^p = (\cos\alpha, \sin\alpha)$ where α the angle between the x-axis and the vector n_i^p . The vector s_i^p, perpendicular to the vector n_i^p, will be defined by $s_i^p = (-\sin\alpha, \cos\alpha)$

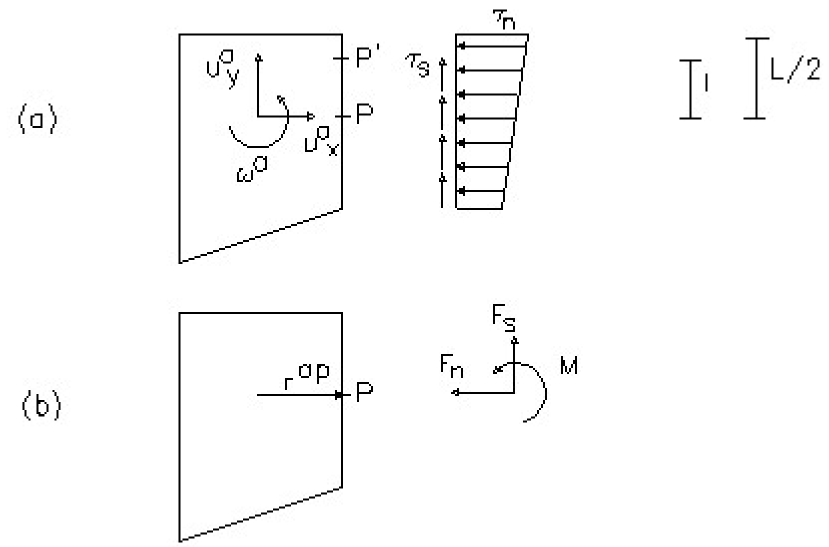


Figure 7.1.1 *(a) shear and normal stresses; (b)Equivalent forces and moments between adjacent slices.*

The displacement vector of the first member of the equation 3 can be transformed from X-Y coordinates in local coordinates n-s as follows:

$$\begin{Bmatrix} \Delta_n^p \\ \Delta_s^p \\ \Delta_\omega^p \end{Bmatrix} = \begin{bmatrix} \cos\alpha & \sin\alpha & 0 \\ -\sin\alpha & \cos\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \cdot \begin{Bmatrix} \Delta_x^p \\ \Delta_y^p \\ \Delta_\omega^p \end{Bmatrix} \qquad (4)$$

Due to the relative movements of two neighboring slices , for a generic point P' of the interface, at a distance l from the central point P like in the figure 7.1.1, the the spring stretch in the normal direction d_n and in the shear direction d_s are given by:

$$d_n = D_n + l \times d_n \qquad d_s = D_s \quad (5)$$

As a result of spring stretch, normal and shear stresses on the slice interface are generated as shown in the figure 7.1.1. These stresses on the interface can be integrated to obtain the resultant forces and moment as follows:

$$F_n = \int_{-L/2}^{L/2} k_n \cdot \delta_n \cdot dl = \int_{-L/2}^{L/2} k_n \cdot \Delta_n \cdot dl + \int_{-L/2}^{L/2} k_n \cdot l \cdot \Delta_\omega \cdot dl \qquad (6)$$

$$F_s = \int_{-L/2}^{L/2} k_s \cdot \delta_s \cdot dl = \int_{-L/2}^{L/2} k_s \cdot \Delta_s \cdot dl \qquad (7)$$

$$M = \int_{-L/2}^{L/2} k_n \cdot l \cdot \delta_s \cdot dl = \int_{-L/2}^{L/2} k_n \cdot l \cdot \Delta_n \cdot dl + \int_{-L/2}^{L/2} k_n \cdot l^2 \cdot \Delta_\omega \cdot dl \quad (8)$$

where

k_n = spring constant per unit length of the normal spring

k_s = spring constant per unit length of the shear spring

L = length of the interface

Note that the springs are elastoplastic so the values of k_n and k_s are function of the deformation, so they must be obtained from stresses-deformations represented in the figure 7.1.2. For non-yield interfaces elastic spring constants k_n and k_s are used. For a yield interface, elastic constants are no longer applicable, and a method that considers the nonlinearity of the problem is required. In this regard, a secant stiffness method is employed. The equivalent spring constants ` $k_n$ and ` k_s for a yield interface can be obtained that corresponds to the deformation of the interface like in the image 7.1.2.

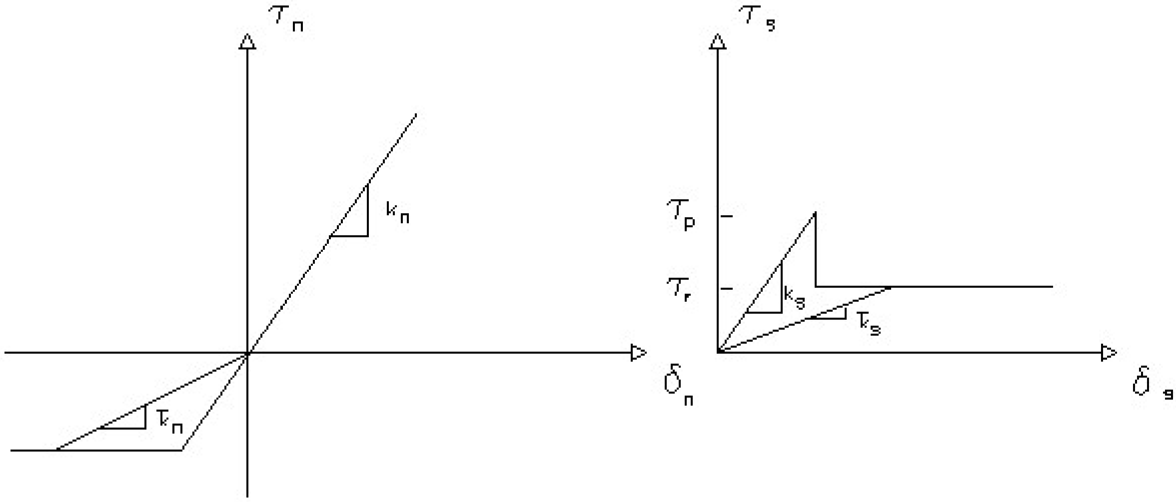


Figure 7.1.2 *Secant stiffness method for obtaining 'K$_n$ 'K$_s$*

Integrating these expressions, since the terms that include the first order K_nL are zero we obtain:

$$F_n = K_n \cdot D_{nx} \cdot L$$
$$F_s = K_s \cdot D_{sx} \cdot L \quad (7')$$
$$M = K_n \cdot \frac{L^3}{12} \quad (8')$$

or, in matrix form:

where:

$$K_n = K_{nx} \cdot L;\ K_s = K_{sx} \cdot L;\ K_w = K_{nx} \cdot \frac{L^3}{12}$$

For convenience, the side forces F_n^P and F_s^P in the local coordinate system *n-s* are transformed to F_x^P and F_y^P in the global coordinate system *X-Y* as shown in the following:

$$\begin{Bmatrix} F_x^p \\ F_y^p \\ M^p \end{Bmatrix} = \begin{bmatrix} \cos\alpha & -\sin\alpha & 0 \\ \sin\alpha & \cos\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \cdot \begin{Bmatrix} F_n^p \\ F_y^p \\ M^p \end{Bmatrix} \qquad (10)$$

The forces acting on all sides of a slice should satisfy the equilibrium requirement, given by:

$$\begin{Bmatrix} F_x^p \\ F_y^p \\ M^p \end{Bmatrix} = \sum_p^N \begin{bmatrix} -1 & 0 & 0 \\ 0 & -1 & 0 \\ r_y^{ap} & -r_x^{ap} & -1 \end{bmatrix} \cdot \begin{Bmatrix} F_x^p \\ F_y^p \\ M^p \end{Bmatrix} \qquad (11)$$

where N is the total number of sides of the slide. The force f_x^a, f_y^a is the weight of the slice acting through its centroid. The body force fxa and moment ma of the slice are usually equal to zero. Combining (3), (4), (9), (10) and (11) we obtain a relationship between the forces and the movements of the slice in the following form:

$$\begin{Bmatrix} f_x^a \\ f_y^a \\ m_\omega^a \end{Bmatrix} = \sum_p^N \begin{bmatrix} c_{11} & c_{12} & c_{13} \\ c_{21} & c_{22} & c_{23} \\ c_{31} & c_{32} & c_{33} \end{bmatrix} \cdot \left\{ \begin{bmatrix} 1 & 0 & -r_y^{bp} \\ 0 & 1 & r_x^{bp} \\ 0 & 0 & 1 \end{bmatrix} \cdot \begin{Bmatrix} u_x^b \\ u_y^b \\ \omega^b \end{Bmatrix} - \begin{bmatrix} 1 & 0 & -r_y^{ap} \\ 0 & 1 & r_x^{ap} \\ 0 & 0 & 1 \end{bmatrix} \cdot \begin{Bmatrix} u_x^a \\ u_y^a \\ \omega^a \end{Bmatrix} \right\} \qquad (1$$

the matrix [c] is given by the multiplication of the following matrices:

Based on (12), three equations of force equilibrium can be set up for each slice, obtaining a system of 3 x N equations for N slices, expressed as follows:

$$\{f\} = [G] \times \{u\} \qquad (14)$$

{ f } : is composed by f_x, f_y and m, for each slice

{ u } : is composed by u_x, u_y ed ω, for each slice

[G]: the total stiffness matrix.

There are six variables for each slice, body forces f_x, f_y , moments m, movement u_x, u_y and rotation ω, in which two body forces and one moment are known: f_x = 0, f_y is the weight of the slice and ω = 0.

Therefore, the set of 3N simultaneous equations can be solved for the 3N unknown variables: u_x, u_y and ω of each slice.

Then the relative movement of two adjacent slices can be determined by (3), and the normal and shear forces between slices can be obtained from the force-displacement relationships (equations 4 and 9).

The normal stress σ_n and shear stress τ_s on the base of each slice can be determined by dividing the force by the area of the base:

$$\sigma_n = \frac{F_n}{L} \qquad (15)$$

$$\tau_s = \frac{F_s}{L} \qquad (16)$$

5 QSIM

5.1 Introduction

QSIM is concerned with the analysis of slope stability in dynamic conditions using Newmark's method.

Newmark's method models a landslide as a rigid block sliding on an inclined plane; movement occurs when horizontal acceleration exceeds the critical value k_c that is calculated by a pseudo static analysis.When acceleration falls below this value motion proceeds with null acceleration.

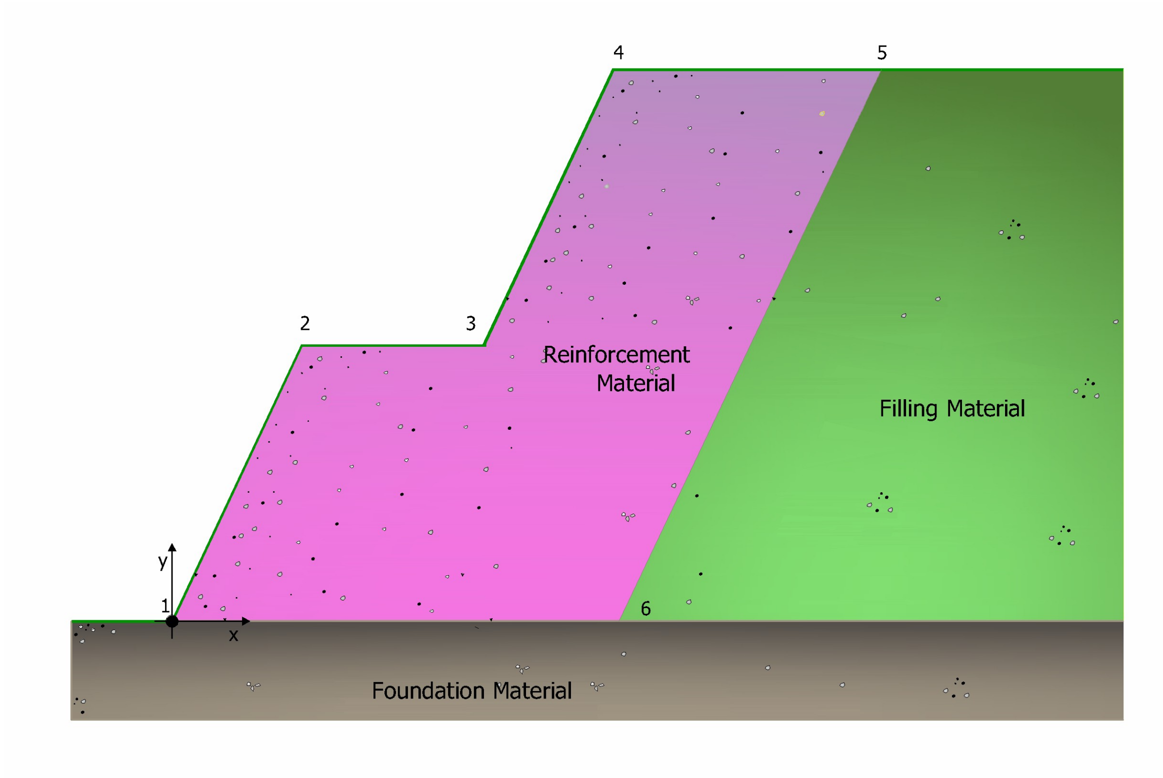
Q_{sim} allows to obtain the trend displacements and velocities during the earthquake and the maximum displacement as well as generate design artificial accelerograms. Other features:

- Import of accelerograms from any external file
- Integration of design accelerograms in the automatic generation
- Computation of response spectrum for design accelerogram
- Computation of response spectra according to normative for ultimate limit state ULS, for damage limit state and for elastic limit state, horizontal and vertical
- Superimposition of the response spectrum of the design accelerogram and normative spectrum to establish compatibility
- Returns the diagrams of accelerations, velocities and displacements
- All graphics can be exported and printed

See also Accelerogram generation

5.1.1 Soil materials

It is possible to assign three different materials:



Reinforcement material

Constitutes the filler material between the reinforcements. Beside unit weight, internal friction angle and cohesion, soil-reinforcement friction angle must be assigned.

Filling material

Constitutes the filler material on the reverse side of the reinforced earth. Beside unit weight, internal friction angle and cohesion, thrust inclination angle must be assigned.

Foundation material

The foundation soil is characterized by: *unit weight, internal friction angle and cohesion*.

5.2 Accelerogram generation

Accelerogram generation

The seismic action that is manifested in a generic site is characterized in a complete known the time history of the accelerations, velocities and displacements of the soil. It is obvious that a knowledge of that detail can not be obtained solely on the basis of macroseismic parameters, such as the magnitude M and the focal position R.

These two parameters, apart from being of physical-empirical nature (unlike the parameters that will be derived from them, which are entirely physical), do not

distinguish the particularities of the various mechanisms that can generate the seismic event. In addition, the local effect is profoundly influenced by the geological and morphological conditions of the portion of crust traversed by the waves, and by the stratigraphic and geotechnical conditions of the site
On the other hand, in the present state of knowledge the two macroseismic parameters are the only ones for which one can obtain a degree of information concretely usable for analysis of seismic risk.
To arrive at the definition of the local seismic motion is ultimately necessary to resort to simplified schemes, in which the macroseismic parameters are integrated with information of empirical nature *(statistical analysis of records of past earthquakes)* or, failing this, with elements based on appropriate considerations of specific data of the problem: distance from potential sources, local characteristics of the soil, etc.
A simplified model of the local seismic motion *(ex., the time history of the acceleration)*, adequate for a number of practical situations, is represented by the expression:

$$a(t) = a \cdot \sum Cn \cdot \cos(\omega_n \cdot t - \phi_n) \qquad (1)$$

where:

- a represents the parameter of intensity, and precisely the peak value of the acceleration of the soil, and is a random variable, whose distribution is obtainable according to the randomness of magnitude M and the focal distance R;
- the terms Cn are the normalized development coefficients in the Fourier summation. They describe the frequency content of the motion, as they provide the relative importance of the different frequency elementary components ω_n

The diagram of the coefficients Cn as a function of the frequencies ω_n represents the *FOURIER* spectrum of the considered seismic event.

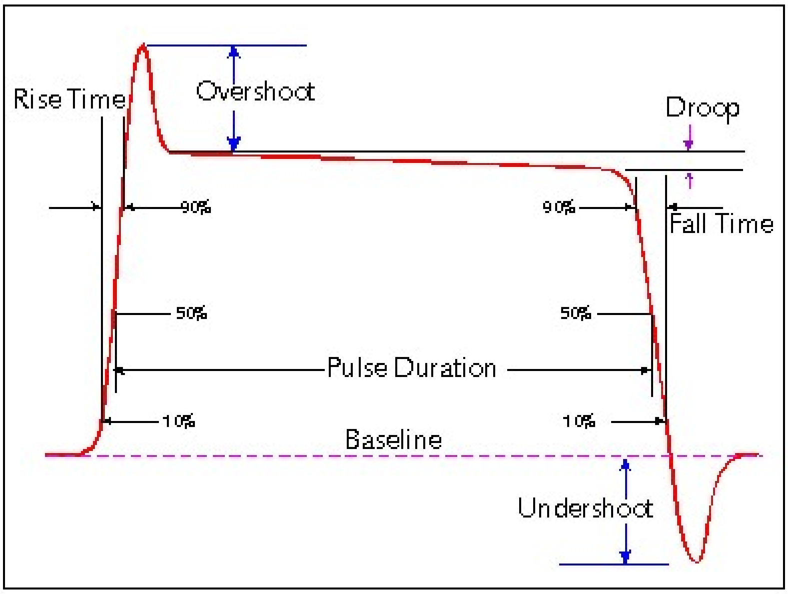
The coefficients Cn are normalized so that the summation in the second member of (1) has a unit maximum value, so that it is:

in accordance with the definition of *a*;

- the terms ω_n=n2π/D are the pulsations (in rad/s) of the various harmonic components, multiple of the minimum frequency: ω_1=2π/ D, where D is the duration of vibration ;
- the terms ϕ_n are the phase angles, one for each harmonic component, between 0 and 2p.

Rise Time

Time to reach maximum acceleration



representative pulse waveform

6 Standards

6.1 Eurocode 7

EN 1997 Eurocode 7 introduces in the verifications regarding structural and geotechnical limit states design approaches that vary for different combinations of groups partial coefficients for actions, for material strength and overall strength of the system.

Each EU member state issues the National Annex (NA) or detailed specifications for the application of the directives contained in EN 1997.

For example, the first approach is used in the UK and Portugal, the second approach in most European countries (Germany, Slovakia, Italy, etc.) for the calculation of the bearing capacity and the third approach in the Netherlands and in most European countries for the calculation of slope stability.

The specifications give the values of the partial factors to be used and indicate approaches to be adopted in the design phase for the different works (bearing capacity, anchors, bulkheads, retaining walls, etc.).

DESIGN APPROACHES

2.4.7.3.4.2 Design Approach 1

1. Except for the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

   Combination 1: **A1“+”M1“+”R1**
   Combination 2: **A2“+”M2“+”R1**

where “+” implies: “to be combined with”.

NOTE: In Combinations 1 and 2, partial factors are applied to actions and to ground strength parameters.

2. For the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:
   Combination 1: **A1“+”M1“+”R1**
   Combination 2: **A2“+”(M1 or M2)“+”R4**

NOTE 1: In Combination 1, partial factors are applied to actions and to ground strength parameters. In Combination 2, partial factors are applied to actions, to ground resistances and sometimes to ground strength parameters.
NOTE 2: In Combination 2, set M1 is used for calculating resistances of piles or anchors and set M2 for calculating unfavorable actions on piles owing e.g. to negative skin friction or transverse loading.

3. If it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out. However, different combinations may be critical to different aspects of the same design.

2.4.7.3.4.3 Design Approach 2

1. It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: **A1"+"M1"+"R2**

NOTE 1: In this approach, partial factors are applied to actions or to the effects of actions and to ground resistances.
NOTE 2: If this approach is used for slope and overall stability analyses the resulting effect of the actions on the failure surface is multiplied by γ_E and the shear resistance along the failure surface is divided by $\gamma_{R;e}$.

2.4.7.3.4.4 Design Approach 3

1. It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: **(A1* or A2†)"+"M2"+"R3**

*on structural actions
†on geotechnical actions

NOTE 1: In this approach, partial factors are applied to actions or the effects of actions from the structure and to ground strength parameters.
NOTE 2: For slope and overall stability analyses, actions on the soil (e.g. structural actions, traffic load) are treated as geotechnical actions by using the set of load factors A2.

The table 3.1. below shows which of partial factor are used in each design approach, depending on the type of structure being designed.

Structure	**Partial factors sets used in Design Approach...**			
	1		**2**	**3**
	Combination 1	**Combination 2**		
General	**A1**+M1+R1	<u>A2</u>+**M2**+R1	**A1+R2**+M1	**A1***(<u>A2</u>$^{+}$)+**M2**+R3
Slope	**A1**+M1+R1	<u>A2</u>+**M2**+R1	**A1**+**R2**+M1	A2+**M2**+R3
Piles and anchor-ages	**A1**+M1+<u>R1</u>	<u>A2</u>+M1+**R4**	**A1**+<u>R2</u>+M1	**A1***(<u>A2</u>$^{+}$)+**M2**+<u>R3</u>

Table 3.1 *- Ultimate limit state, design approach (*on structural actions,+ on geotechnical actions)*

Design Approach 1	**Combination 1**			**Combination 2**		
	A1	**M1**	**R1**	**A2**	**M2**	**R1**

Permanent actions (G)	Unfavorable	γ_G	1,35			1,0		
	Favorable	$\gamma_{G,fav}$	1,0			1,0		
Variable actions (Q)	Unfavorable	γ_Q	1,5			1,3		
	Favorable	$\gamma_{Q,fav}$	0			0		
Coeff.of schearing resistance (tanφ)		γ_φ		1,0			1,25	
Effective cohesion (c')		$\gamma_{c'}$		1,0			1,25	
Undrained strength (cu)		γ_{cu}		1,0			1,4	
Unconfined compressive strength (q_u)		γ_{qu}		1,0			1,4	
Weight density (γ)		γ_γ		1,0			1,0	
Resistance (R)		γ_R			1,0			1,0

Table 3.2 - *Shows the relative magnitude of the key parameters when using Combination and using Combination 2*

Design Approach 2

			A1	M1	R1
Permanent actions (G)	Unfavorable	γ_G	1,35		
	Favorable	$\gamma_{G,fav}$	1,0		
Variable actions (Q)	Unfavorable	γ_Q	1,5		
	Favorable	$\gamma_{Q,fav}$	0		
Material properties(c)		γ_M		1,0	
Material resistance (Rv)		γ_{Rv}			1,4
Sliding resistance (Rh)		γ_{Rh}			1,1
Earth resistance against retaining structures		γ_{Re}			1,4
....in slope					1,1

***Table 3.3** - Shows the relative magnitude of the key parameters when using Design Approach 2*

Design Approach 3

			A1	A2	M2	R3
Permanent actions (G)	Unfavorable	γ_G	1,35	1,0		
	Favorable	$\gamma_{G,fav}$	1,0	1,0		
Variable actions (Q)	Unfavorable	γ_Q	1,5	1,3		
	Favorable	$\gamma_{Q,fav}$	0	0		
Coeff.of schearing resistance (tanφ)		γ_φ			1,25	
Effective cohesion (c')		$\gamma_{c'}$			1,25	
Undrained strength (cu)		γ_{cu}			1,4	
Unconfined compressive strength (q_u)		γ_{qu}			1,4	
Weight density (γ)		γ_g			1,0	
Resistance (R) (except for pile shaft in tension)		γ_R				1,0
Pile shatf resistance in tension		$\gamma_{R,st}$				1,1

***Table 3.4** - Shows the relative magnitude of the key parameters when using Design Approach 3*

Spread foundations

6.1 General

1. The provisions of this Section apply to spread foundations including pads, strips and rafts.
2. Some of the provisions may be applied to deep foundations such as caissons.

6.2 Limit states

1. The following limit states shall be considered and an appropriate list shall be compiled:

- loss of overall stability;
- bearing resistance failure, punching failure, squeezing;
- failure by sliding;
- combined failure in the ground and in the structure;
- structural failure due to foundation movement;
- excessive settlements;
- excessive heave due to swelling, frost and other causes;
- unacceptable vibrations.

6.3 Actions and design situations

1. Design situations shall be selected in accordance with 2.2.
2. The actions listed in 2.4.2(4) should be considered when selecting the limit states for calculation.
3. If structural stiffness is significant, an analysis of the interaction between the structure and the ground should be performed in order to determine the distribution of actions.

6.4 Design and construction considerations

1. When choosing the depth of a spread foundation the following shall be considered:
   - reaching an adequate bearing stratum;
   - the depth above which shrinkage and swelling of clay soils, due to seasonal weather
     changes, or to trees and shrubs, may cause appreciable movements;
   - the depth above which frost damage may occur;
   - the level of the water table in the ground and the problems, which may occur if excavation
     for the foundation is required below this level;
   - possible ground movements and reductions in the strength of the bearing stratum by
     seepage or climatic effects or by construction procedures;
   - the effects of excavations on nearby foundations and structures;
   - anticipated excavations for services close to the foundation;
   - high or low temperatures transmitted from the building;
   - the possibility of scour;

- the effects of variation of water content due to long periods of drought, and subsequent
  periods of rain, on the properties of volume-unstable soils in arid climatic areas;
- the presence of soluble materials, e.g. limestone, claystone, gypsum, salt rocks;

2. Frost damage will not occur if:
   - the soil is not frost-susceptible;
   - the foundation level is beneath frost-free depth;
   - frost is eliminated by insulation.
3. EN-ISO 13793:2001 may be applied for frost protecting measures for building foundations.
4. In addition to fulfilling the performance requirements, the design foundation width shall take account of practical considerations such as economic excavation, setting out tolerances, working space requirements and the dimensions of the wall or column supported by the foundation.
5. One of the following design methods shall be used for spread foundations:
   - a direct method, in which separate analyses are carried out for each limit state. When checking against an ultimate limit state, the calculation shall model as closely as possible the failure mechanism, which is envisaged. When checking against a serviceability limit
   state, a settlement calculation shall be used;
   - an indirect method using comparable experience and the results of field or laboratory measurements or observations, and chosen in relation to serviceability limit state loads so as to satisfy the requirements of all relevant limit states;
   - a prescriptive method in which a presumed bearing resistance is used (see 2.5).
6. Calculation models for ultimate and serviceability limit state design of spread foundations on soil given in 6.5 and 6.6 respectively should be applied. Presumed bearing pressures for the design of spread foundations on rock should be applied according to 6.7.

6.5 Ultimate limit state design

6.5.1 Overall stability

1. Overall stability, with or without the foundations, shall be checked particularly in the following situations:
   - near or on a natural or man-made slope;
   - near an excavation or a retaining wall;
   - near a river, a canal, a lake, a reservoir or the sea shore;
   - near mine workings or buried structures.

2. For such situations, it shall be demonstrated using the principles described in Section 11, that a stability failure of the ground mass containing the foundation is sufficiently improbable.

6.5.2 Bearing resistance

6.5.2.1 General

1. The following inequality shall be satisfied for all ultimate limit states:

$$V_d \leq R_d \qquad [6.1]$$

2. R_d shall be calculated according to 2.4.
3. V_d shall include the weight of the foundation, the weight of any backfill material and all earth pressures, either favorable or unfavorable. Water pressures not caused by the foundation load shall be included as actions.

6.5.2.2 Analytical method

1. The sample analytical calculation for bearing resistance given in Annex D may be used.
2. An analytical evaluation of the short-term and long-term values of R_d shall be considered, particularly in fine-grained soils.
3. Where the soil or rock mass beneath a foundation presents a definite structural pattern of layering or other discontinuities, the assumed rupture mechanism and the selected shear strength and deformation parameters shall take into account the structural characteristics of the ground.
4. When calculating the design bearing resistance of a foundation on layered deposits, the properties of which vary greatly between one another, the design values of the ground parameters shall be determined for each layer.
5. Where a strong formation underlies a weak formation, the bearing resistance may be calculated using the shear strength parameters of the weak formation. For the reverse situation, punching failure should be checked.
6. Analytical methods are often not applicable to the design situations described in 6.5.2.2(3)P, 6.5.2.2(4)P and 6.5.2.2(5). Numerical procedures should then be applied to determine the most unfavorable failure mechanism.
7. The overall stability calculations described in Section 11 may be applied.

6.5.2.3 Semi-empirical method

1. The sample semi-empirical method for bearing resistance estimation using pressuremeter test results given in Annex E is recommended.

6.5.2.4 Prescriptive method using presumed bearing resistance

1. The sample method for deriving the presumed bearing resistance for spread foundations on rock given in Annex G is recommended. When this method is applied, the design result should be evaluated on the basis of comparable experience.

6.5.3 Sliding resistance

1. Where the loading is not normal to the foundation base, foundations shall be checked against failure by sliding on the base.
2. The following inequality shall be satisfied:

$$H_d \leq S_d + E_{pd} \qquad [6.2]$$

3. H_d shall include the design values of any active earth forces imposed on the foundation.
4. R_d shall be calculated according to 2.4.
5. The values of R_d and R_p; d should be related to the scale of movement anticipated under the limit state of loading considered. For large movements, the possible relevance of post-peak behavior should be considered. The value of $R_{p;d}$ selected should reflect the anticipated life of the structure.
6. For foundations bearing within the zone of seasonal movements of clay soils, the possibility that the clay could shrink away from the vertical faces of foundations shall be considered.
7. The possibility that the soil in front of the foundation may be removed by erosion or human activity shall be considered.
8. For drained conditions, the design shear resistance, R_d , shall be calculated either by factoring the ground properties or the ground resistance as follows;

$$S_d = V'_d \tan \delta_d \qquad [6.3]$$

or

$$R_d = (V'_d \tan \delta_k) / \gamma_{R;h} \ (6.3b)$$

Note: In design procedures where the effects of actions are factored, the partial factor for the actions (γ_F) is 1,0 and $V'_d = V'_k$ in equation (6.3b).

9. In determining V_d', account shall be taken of whether H_d and V'_d are dependent or independent actions.
10. The design friction angle δ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance, $?'_{cv;d}$, for cast-in-situ concrete

foundations and equal to 2/3 ?'$_{cv;d}$ for smooth precast foundations. Any effective cohesion c' should be neglected.

11. For undrained conditions, the design shearing resistance, R_d , shall be calculated either by factoring the ground properties or the ground resistance as follows:

$$S_d = A' c_{u;d} \qquad [6.4a]$$

or

$$R_d = (Ac_{u;k}) / \gamma_{R;h} \; (6.4b)$$

12. If it is possible for water or air to reach the interface between a foundation and an undrained clay subgrade, the following check shall be made:

$$R_d \leq 0{,}4\ V_d \qquad [6.5]$$

13. Requirement (6.5) may only be disregarded if the formation of a gap between the foundation and the ground will be prevented by suction in areas where there is no positive bearing pressure.

6.5.4 Loads with large eccentricities

1. Special precautions shall be taken where the eccentricity of loading exceeds 1/3 of the width of a rectangular footing or 0,6 of the radius of a circular footing. Such precautions include:

- careful review of the design values of actions in accordance with 2.4.2;
- designing the location of the foundation edge by taking into account the magnitude of construction tolerances.

2. Unless special care is taken during the works, tolerances up to 0,10 m should be considered.

6.5.5 Structural failure due to foundation movement

1. Differential vertical and horizontal foundation displacements shall be considered to ensure that they do not lead to an ultimate limit state occurring in the supported structure.
2. A presumed bearing pressure may be adopted (see 2.5) provided displacements will not cause an ultimate limit state in the structure.
3. In ground that may swell, the potential differential heave shall be assessed and the foundations and structure designed to resist or accommodate it.

6.6 Serviceability limit state design

6.6.1 General

1. Account shall be taken of displacements caused by actions on the foundation, such as those listed in 2.4.2(4).
2. In assessing the magnitude of foundation displacements, account shall be taken of comparable experience, as defined in 1.5.2.2. If necessary, calculations of displacements shall also be carried out.
3. For soft clays, settlement calculations shall always be carried out.
4. For spread foundations on stiff and firm clays in Geotechnical Categories 2 and 3, calculations of vertical displacement (settlement) should usually be undertaken. Methods that may be used to calculate settlements caused by loads on the foundation are given in 6.6.2.
5. The serviceability limit state design loads shall be used when calculating foundation displacements for comparison with serviceability criteria.
6. Calculations of settlements should not be regarded as accurate. They merely provide an approximate indication.
7. Foundation displacements shall be considered both in terms of displacement of the entire foundation and differential displacements of parts of the foundation.
8. The effect of neighboring foundations and fills shall be taken into account when calculating the stress increase in the ground and its influence on ground compressibility.
9. The possible range of relative rotations of the foundation shall be assessed and compared with the relevant limiting values for movements discussed in 2.4.9.

6.6.2 Settlement

1. Calculations of settlements shall include both immediate and delayed settlement.
2. The following three components of settlement should be considered for partially or fully saturated soils:

   - s0 : immediate settlement; for fully-saturated soil due to shear deformation at constant volume, and for partially-saturated soil due to both shear deformation and volume reduction;
   - s1 : settlement caused by consolidation;
   - s2 : settlement caused by creep.

3. The sample methods for evaluating settlements s_0 and s_1 given in Annex F may be applied.
4. Special consideration should be given to soils such as organic soils and soft clays, in which settlement may be prolonged almost indefinitely due to creep.

5. The depth of the compressible soil layer to be considered when calculating settlement should depend on the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements.
6. This depth may normally be taken as the depth at which the effective vertical stress due to the foundation load is 20 % of the effective overburden stress.
7. For many cases this depth may also be roughly estimated as 1 to 2 times the foundation width, but may be reduced for lightly-loaded, wider foundation rafts.

   Note: This approach is not valid for very soft soils.
8. Any possible additional settlement caused by self-weight compaction of the soil shall be assessed.
9. The following should be considered:

   - the possible effects of self-weight, flooding and vibration on fill and collapsible soils;
   - the effects of stress changes on crushable sands.
10. Either linear or non-linear models of the ground stiffness shall be adopted, as appropriate.
11. To ensure the avoidance of a serviceability limit state, assessment of differential settlements and relative rotations shall take account of both the distribution of loads and the possible variability of the ground.
12. Differential settlement calculations that ignore the stiffness of the structure tend to be over-predictions. An analysis of ground-structure interaction may be used to justify reduced values of differential settlements.
13. Allowance should be made for differential settlement caused by variability of the ground unless it is prevented by the stiffness of the structure.
14. For spread foundations on natural ground, it should be taken into account that some differential settlement normally occurs even if the calculation predicts uniform settlement only.
15. The tilting of an eccentrically loaded foundation should be estimated by assuming a linear bearing pressure distribution and then calculating the settlement at the corner points of the foundation, using the vertical stress distribution in the ground beneath each corner point and the settlement calculation methods described above.
16. For conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained shear strength, to the applied serviceability loading should be calculated (see 2.4.8(4)). If this ratio is less than 3, calculations of settlements should always be undertaken. If the ratio is less than 2, the calculations should take account of non-linear stiffness effects in the ground.

6.2 Eurocode 8

3 GROUND CONDITIONS AND SEISMIC ACTION (EC8 - part 1)

3.1 Ground conditions

3.1.2 Identification of ground types

1. Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 3.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

> **Note:** The ground classification scheme accounting for deep geology for use in a country may be specified in its National Annex, including the values of the parameters S, T_B, T_C and T_D defining the horizontal and vertical elastic response spectra in accordance with **3.2.2.2** and **3.2.2.3.**

Ground type	Description of stratigraphic profile	$V_{s,30}$ (m/s)	N_{SPT} (blows/30 cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800		
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with *v* s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	<100 (indicative)		10-20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Prospect 3.1*-Ground types*

2. The site should be classified according to the value of the average shear wave velocity, $v_{s,30}$, if this is available. Otherwise the value of N_{SPT} should be used.

3. The average shear wave velocity $v_{s,30}$ should be computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_i}} \qquad (3.1)$$

where h_i and v_i denote the thickness (in meters) and shear-wave velocity (at a shear strain level of 10^{-5} or less) of the *i*-the formation or layer, in a total of *N*, existing in the top 30 m.

4. For sites with ground conditions matching either one of the two special ground types S_1 or S_2 , special studies for the definition of the seismic action are required. For these types, and particularly for S_2, the possibility of soil failure under the seismic action shall be taken into account.

Note: Special attention should be paid if the deposit is of ground type S_1 . Such soils typically have very low values of v_s, low internal damping and an abnormally extended range of linear behavior and can therefore produce anomalous seismic site amplification and soil-structure interaction effects (see EN 1998-5:2004, Section 6). In this case, a special study to define the seismic action should be carried out, in order to establish the dependence of the response
spectrum on the thickness and v_s value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

3.2 Seismic action

3.2.1 Seismic zones

1. For the purpose of EN 1998, national territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant.
2. For most of the applications of EN 1998, the hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_{gR} . Additional parameters required for specific types of structures are given in the relevant Parts of EN 1998.

Note: The reference peak ground acceleration on type A ground, a_{gR} , for use in a country or parts of the country, may be derived from zonation maps found in its National Annex.

3. The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period T_{NCR} of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, P_{NCR}) chosen by the National Authorities (see 2.1(1)P). An importance factor γ_1 equal to 1,0 is assigned to this reference return period. For return periods other than the reference (see importance classes in 2.1(3)P and (4)), the design ground acceleration on type A ground a_g is equal to a_{gR} times the importance factor γ_I ($a_g = \gamma_1 \; x \; a_{gR}$). (See Note to 2.1(4)).
4. In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used.

   Note: The selection of the categories of structures, ground types and seismic zones in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended to consider as low seismicity cases either those in which the design ground acceleration on type A ground, a_g , is not greater than 0,08g (0,78 m/s^2), or those where the product a_g x S is not greater than 0,1 g (0,98 m/s^2). The selection of whether the value of a_g , or that of the product a_g x S will be used in a country to define the threshold for low seismicity cases, may be found in its National Annex.

5. In cases of very low seismicity, the provisions of EN 1998 need not be observed.

   Note: The selection of the categories of structures, ground types and seismic zones in a country for which the EN 1998 provisions need not be observed (cases of very low seismicity) may be found in its National Annex. It is recommended to consider as very low seismicity cases either those in which the design ground acceleration on type A ground, a_g , is not greater than 0,04g (0,39 m/s^2), or those where the product a_g x S is not greater than 0,05g (0,49 m/s^2). The selection of whether the value of a_g , or that of the product a_g x S will be used in a country to define the threshold for very low seismicity cases, can be found in its National Annex.

3.2.2 Basic representation of the seismic action

3.2.2.1 General

1. Within the scope of EN 1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an ”elastic response spectrum”.

2. The shape of the elastic response spectrum is taken as being the same for the two levels of seismic action introduced in 2.1(1)P and 2.2.1(1)P for the no-collapse requirement (ultimate limit state – design seismic action) and for the damage limitation requirement.
3. The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum.
4. For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquake magnitudes generated from them.

3.2.2.2 Horizontal elastic response spectrum

1. For the horizontal components of the seismic action, the elastic response spectrum $S_e(T)$ is defined by the following expressions (see Figure. 3.1):

$$0 \le T \le T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right] \qquad (3.2)$$

$$T_B \le T \le T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \qquad (3.3)$$

$$T_C \le T \le T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \frac{T}{T_B} \qquad (3.4)$$

$$T_D \le T \le 4(s) : S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \left[\frac{T_C \cdot T_D}{T}\right] \qquad (3.5)$$

where:

$S_e(T)$	is the elastic response spectrum
T	is the vibration period of a linear single-degree-of-freedom system
a_g	is the design ground acceleration on type A ground ($a_g = \gamma_1 * a_g R$);
T_B	is the lower limit of the period of the constant spectral acceleration branch
T_C	is the upper limit of the period of the constant spectral acceleration branch;
T_D	is the value defining the beginning of the constant displacement response range of the spectrum
S	is the soil factor;

η is the damping correction factor with a reference value of η= 1 for 5% viscous damping, see (3) of this subclause.

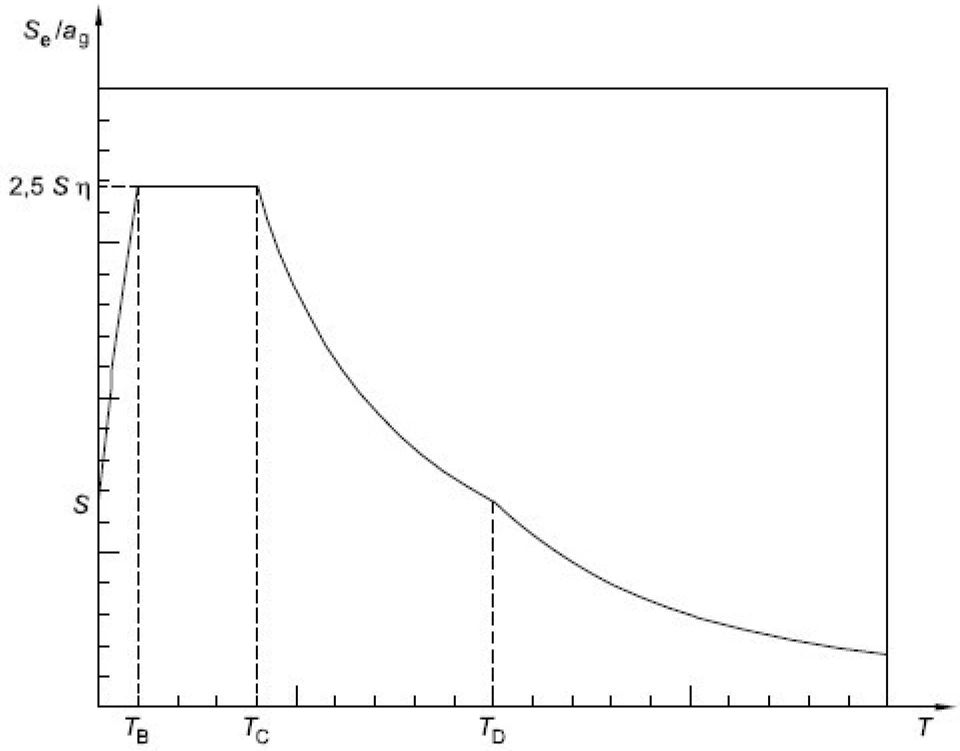


Figure 3.1 *- Shape of the elastic response spectrum*

2. The values of the periods T_B , T_C and T_D and of the soil factor *S* describing the shape of the elastic response spectrum depend upon the ground type.

Note 1: The values to be ascribed to T_B, T_C and T_D and S for each ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If deep geology is not accounted for (see 3.1.2(1)), the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters S, T_B, T_C and T_D are given in Table 3.2 for the Type 1 Spectrum and in Table 3.3 for the Type 2 Spectrum. Figure 3.2 and Figure 3.3 show the shapes of the recommended Type 1 and Type 2 spectra, respectively, normalized by a_g, for 5% damping. Different spectra may be defined in the National Annex, if deep geology is accounted for.

Ground type	S	$T_B(s)$	$T_C(s)$	$T_D(s)$
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,15	2,0

Table 3.2 - *Values of the parameters describing the recommended Type 1 elastic response spectra*

Ground type	S	$T_B(s)$	$T_C(s)$	$T_D(s)$
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Table 3.3 - *Values of the parameters describing the recommended Type 2 elastic response spectra*

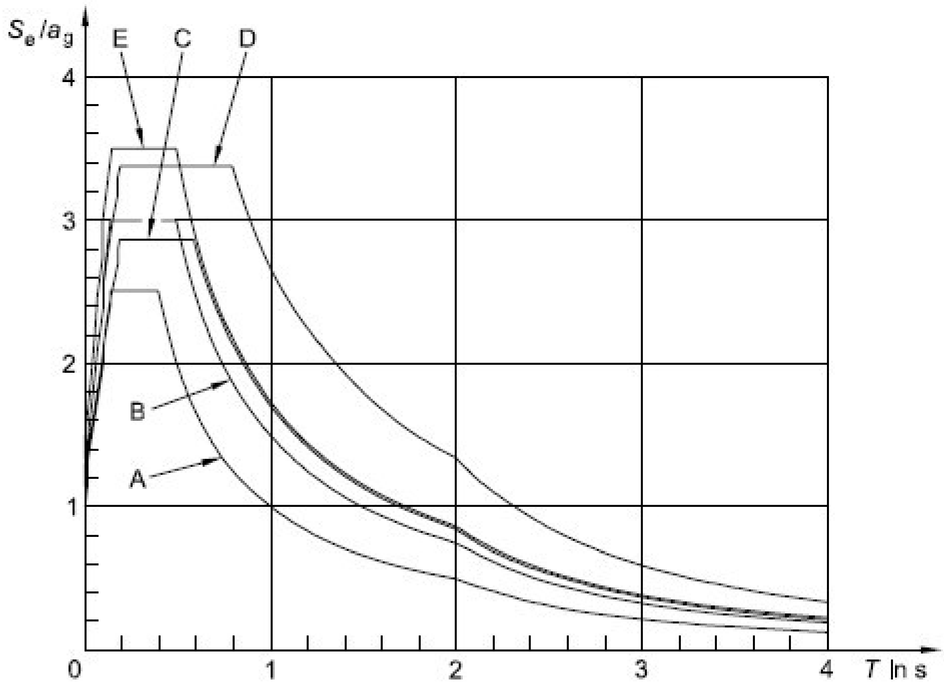


Figure 3.2 - *Recommended Type 1 elastic response spectra for ground types A to E (5% damping)*

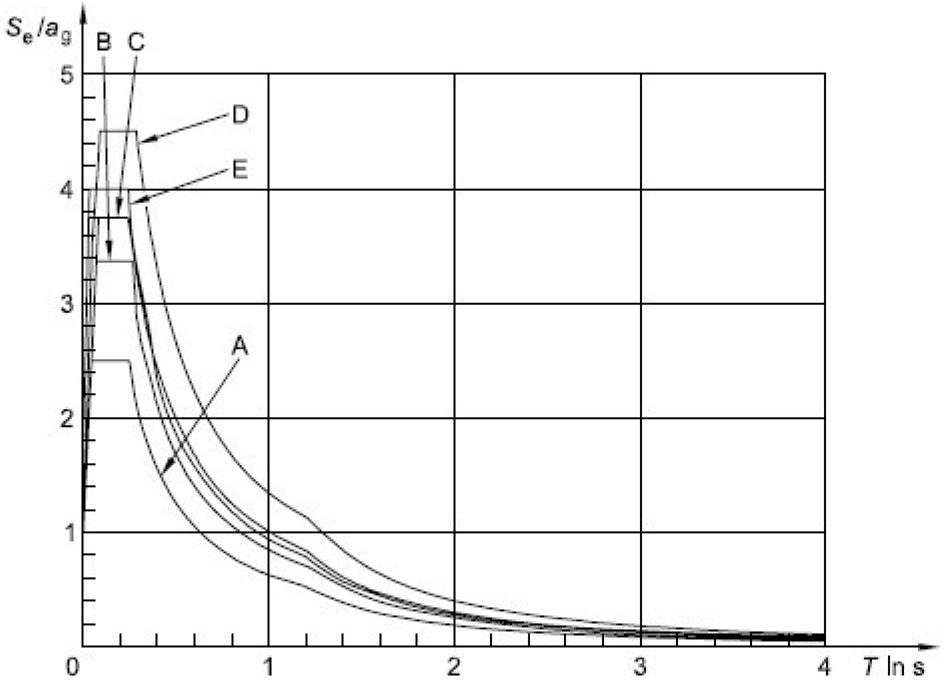


Figure 3.3 - *Recommended Type 2 elastic response spectra for ground types A to E (5% damping)*

Note 2: For ground types S_1 and S_2, special studies should provide the corresponding values of S, T_B , T_C and T_D.

3. The value of the damping correction factor η may be determined by the expression:

$$\eta = \sqrt{10/(5+\xi)} \geq 0{,}55 \qquad (3.6)$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage.

4. If for special cases a viscous damping ratio different from 5% is to be used, this value is given in the relevant Part of EN 1998.
5. The elastic displacement response spectrum, $S_{De}(T)$, shall be obtained by direct transformation of the elastic acceleration response spectrum, $S_e(T)$, using the following expression:

(3.7)

6. Expression (3.7) should normally be applied for vibration periods not exceeding 4,0 s. For structures with vibration periods longer than 4,0 s, a more complete definition of the elastic displacement spectrum is possible.

Note: For the Type 1 elastic response spectrum referred to in Note 1 to 3.2.2.2(2)P, such a definition is presented in Informative Annex A in terms of the displacement response spectrum. For periods longer than 4,0 s, the elastic acceleration response spectrum may be derived from the elastic displacement response spectrum by inverting expression (3.7).

3.2.2.3 Vertical elastic response spectrum

1. The vertical component of the seismic action shall be represented by an elastic response spectrum, $S_{ve}(T)$, derived using expressions (3.8)-(3.11).

Note: The values to be ascribed to T_B , T_C , T_D and a_{vg} for each type (shape) of vertical spectrum to be used in a country may be found in its National Annex. The recommended choice is the use of two types of vertical spectra: Type 1 and Type 2. As for the spectra defining the horizontal components of the seismic action, if the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters describing the vertical spectra are given in Table 3.4. These recommended values do not apply for special ground types S_1 and S_2.

$$0 \le T \le T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1) \right] \qquad (3.8)$$

$$T_B \le T \le T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \qquad (3.9)$$

$$T_C \le T \le T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3{,}0 \cdot \left[\frac{T_C}{T} \right] \qquad (3.10)$$

$$T_D \le T \le 4(s) : S_{ve}(T) = a_g \cdot \eta \cdot 3{,}0 \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \qquad (3.11)$$

Spectrum	a_{vg}/a_g	T_B(s)	T_C(s)	T_D(s)
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

***Table 3.4** - Recommended values of parameters describing the vertical elastic response spectra*

2. To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behavior factor q.

3. The behavior factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behavior factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998. The value of the behavior factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

4. For the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \le T \le T_B : S_{ve}(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3}\right)\right] \qquad (3.13)$$

$$T_B \le T \le T_C : S_d(T) = a_{vg} \cdot S \cdot \frac{2{,}5}{q} \qquad (3.14)$$

$$T_C \le T \le T_D : S_d(T) = \begin{cases} = a_g \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_C}{T}\right] \\ \ge \beta \cdot a_g \end{cases} \qquad (3.15)$$

$$T_D \le T : S_d(T) = \begin{cases} = a_g \cdot S \cdot \dfrac{2{,}5}{q} \cdot \left[\dfrac{T_C \cdot T_D}{T^2}\right] \\ \ge \beta \cdot a_g \end{cases} \qquad (3.16)$$

where:

a_g, S, T_C ,T_D	are as defined in 3.2.2.2;
$S_d(T)$	is the design spectrum;
q	is the behavior factor;
β	is the lower bound factor for the horizontal design spectrum.

Note: The value to be ascribed to β for use in a country can be found in its National Annex. The recommended value for β is 0,2.

5. For the vertical component of the seismic action the design spectrum is given by expressions (3.13) to (3.16), with the design ground acceleration in the vertical direction, a_{vg} replacing a_g , S taken as being equal to 1,0 and the other parameters as defined in 3.2.2.3.
6. For the vertical component of the seismic action a behavior factor q up to to 1,5 should generally be adopted for all materials and structural systems.
7. The adoption of values for q greater than 1,5 in the vertical direction should be justified through an appropriate analysis.
8. The design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation systems.

3.2.3 Alternative representations of the seismic action

3.2.3.1 Time - history representation

3.2.3.1.1 General

1. The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement)
2. When a spatial model is required, the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions. Simplifications are possible in accordance with the relevant Parts of EN 1998.
3. Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see 3.2.3.1.2) and recorded or simulated accelerograms (see 3.2.3.1.3).

3.2.3.1.2 Artificial accelerograms

1. Artificial accelerograms shall be generated so as to match the elastic response spectra given in 3.2.2.2 and 3.2.2.3 for 5% viscous damping (ξ= 5%).
2. The duration of the accelerograms shall be consistent with the magnitude and the other relevant features of the seismic event underlying the establishment of a_g .
3. When site-specific data are not available, the minimum duration T_s of the stationary part of the accelerograms should be equal to 10 s.
4. The suite of artificial accelerograms should observe the following rules:

   a) a minimum of 3 accelerograms should be used;

   b) the mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of $a_g S$ for the site in question.

   c) in the range of periods between $0,2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

3.2.3.1.3 Recorded or simulated accelerograms

1. Recorded accelerograms, or accelerograms generated through a physical simulation of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of a $a_g S$ for the zone under consideration.
2. For soil amplification analyses and for dynamic slope stability verifications see EN 1998-5:2004, 2.2.
3. The suite of recorded or simulated accelerograms to be used should satisfy 3.2.3.1.2(4).

3.2.3.2 Spatial model of the seismic action

1. For structures with special characteristics such that the assumption of the same excitation at all support points cannot reasonably be made, spatial models of the seismic action shall be used (see 3.2.2.1(8)).

2. Such spatial models shall be consistent with the elastic response spectra used for the basic definition of the seismic action in accordance with 3.2.2.2 and 3.2.2.3.

3.2.4 Combinations of the seismic action with other actions

1. The design value E_d of the effects of actions in the seismic design situation shall be determined in accordance with EN 1990:2002, 6.4.3.4.
2. The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\sum G_{k,j} \text{"+"} \sum \Psi_{E,i} \cdot Q_{k,i} \qquad (3.17)$$

where:

$\psi_{E,i}$ is the combination coefficient for variable action *i* (see 4.2.4).

3. The combination coefficients $\psi_{E,i}$ take into account the likelihood of the loads $Q_{k,i}$ not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.
4. Values of $\Psi_{E,i}$ are given in EN 1990:2002 and values of $\Psi_{E,i}$ other types of structures are given in the relevant parts of EN 1998.

4.1.3 Slope stability

4.1.3.3 Methods of analysis (EC 8-part 5)

1. The response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods subject to the limitations of (3) and (8) of this subclause.
2. In modelling the mechanical behavior of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading shall be taken into account.
3. The stability verification may be carried out by means of simplified pseudostatic methods where the surface topography and soil stratigraphy do not present very abrupt irregularities.
4. The pseudo-static methods of stability analysis are similar to those indicated in EN 1997-1:2004, 11.5, except for the inclusion of horizontal and vertical inertia forces

applied to every portion of the soil mass and to any gravity loads acting on top of the slope.

5. The design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

F_H = 0,5 a S_w
F_V = ± 0,5 F_H if the ratio a_{vg}/a_g is greater than 0,6
F_V = ± 0,33 F_H if the ratio a_{vg}/a_g is not greater than 0,6.

Where:
a is the ratio of the design ground acceleration on type A ground, a_g , to the acceleration of gravity g;
a_{vg} is the design ground acceleration in the vertical direction;
a_g is the design ground acceleration for type A ground;
S is the soil parameter of EN 1998-1:2004, 3.2.2.2;
W is the weight of the sliding mass.

A topographic amplification factor for a g shall be taken into account according to 4.1.3.2 (2).

6. A limit state condition shall then be checked for the least safe potential slip surface.
7. The serviceability limit state condition may be checked by calculating the permanent displacement of the sliding mass by using a simplified dynamic model consisting of a rigid block sliding against a friction force on the slope. In this model the seismic action should be a time history representation in accordance with 2.2 and based on the design acceleration without reductions.
8. Simplified methods, such as the pseudo-static simplified methods mentioned in (3) to (6) in this subclause, shall not be used for soils capable of developing high pore water pressures or significant degradation of stiffness under cyclic loading.
9. The pore pressure increment should be evaluated using appropriate tests. In the absence of such tests, and for the purpose of preliminary design, it may be estimated through empirical correlations.

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8 Geoapp

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8.1 Geoapp Section

General and Engineering, Geotechnics and Geology

Among the applications present, a wide range can be used for **Slope**. For this purpose, the following applications are recommended:

- ➢ Sismogenetic zone
- ➢ Soil classification SMC
- ➢ Seismic parameters
- ➢ NSPT
- ➢ Slope stability
- ➢ Landslide trigger
- ➢ Critical high (maximum depth that can be excavated without failure)
- ➢ Bearing capacity
- ➢ Lithostatic tensions

➢ Foundation piles, horizontal reaction coefficient

9 Recommended books

Geotechnical, engineering, and geology books

Portal books: explore the library

• Methods for estimating the geotechnical properties of the soil

Methods for estimating the geotechnical properties of the soil: semi-empirical correlations of geotechnical parameters based on in-situ soil tests.

This text is designed for all professionals who operate in the geotechnical subsurface investigation. The purpose of this text is to provide an easy reference tool relatively to the means available today.

Theoretical insights have been avoided, for which please refer to the bibliography attached, except in cases where these were considered essential for the understanding of the formulation. The reason for this is obvious: make the text as easy to read as possible.

After a brief introduction about volumetric and density relationships with the most common definitions used for soils, in the following chapters we briefly described some of the most widespread in situ geotechnical testing and correlations to derive empirically geotechnical parameters and a number of useful formulations available today in the field of Geology.

The text concludes with the inclusion of formulas used in Technical Geology, considered of daily use to those working in the sector.

The topics are intended to provide a basic understanding of the in situ geotechnical testing and evaluation of geotechnical parameters necessary to define the geotechnical model.

10 Utility

10.1 Database of soil physical characteristics

Approximate values of the tangential restitution coefficient (R_t) for the various morphological categories

MORPHOLOGY	R_t
Bedrock	0.87
Outcrops of rock debris	0.85
Coarse debris not vegetated	0.85
Average debris not vegetated	0.83
Vegetated debris with shrubs	0.70
Vegetated debris in forest	0.60
Bare soil or lawn	0.55
Paved surfaces	0.90

Approximate values of the normal restitution coefficient (R_n) for the various morphological categories

MORPHOLOGY	R_n
Bedrock	0.40
Outcrops of rock debris	0.38
Coarse debris non-vegetated	0.35
Average debris not vegetated	0.31
Vegetated debris with shrubs	0.30
Vegetated debris in forest	0.28
Bare soil or lawn	0.25
Paved surfaces	0.40

Approximate values of the unit weight in Kg/m^3

Soil	Minimum value	Maximum value
Dry gravel	1800	2000
Wet gravel	1900	2100
Compact dry sand	1700	2000
Compact wet sand	1900	2100
Loose dry sand	1500	1800
Loose wet sand	1600	1900
Sandy clay	1800	2200
Hard clay	2000	2100
Semisolid clay	1900	1950
Soft clay	1800	1850
Peat	1000	1100

Approximate values of the friction angle, in degrees, for soils

Soil	Minimum value	Maximum value
Compact gravel	35	35
Loose gravel	34	35
Compact sand	35	45
Loose sand	25	35
Sandy marl	22	29
Fat marl	16	22
Fat clay	0	30
Sandy clay	16	28
Silt	20	27

Approximate values of cohesion in Kg/cm^2

Soil	Value
Sandy clay	0.20
Soft clay	0.10

Soil	Value
Plastic clay	0.25
Semisolid clay	0.50
Solid clay	1
Tenacious clay	2÷10
Compact silt	0.10

Approximate values of the elastic module, in Kg/cm^2

Soil	Maximum value of E	Minimum value of E
Very soft clay	153	20.4
Soft clay	255	51
Medium clay	510	153
Hard clay	1020	510
Sandy clay	2550	255
Loess	612	153
Silty sand	204	51
Loose sand	255	102
Compact sand	816	510
Clayey schist	51000	1530
Silt	204	20.4
Loose sand and gravel	1530	510
Compact sand and gravel	2040	1020

Approximate values of the Poisson ratio for soils

Soil	Maximum value of n	Minimum value of n
Saturated clay	0.5	0.4
Not saturated clay	0.3	0.1
Sandy clay	0.3	0.2
Silt	0.35	0.3
Sand	1.0	-0.1
Gravelly sand commonly used	0.4	0.3
Loess	0.3	0.1
Ice	0.36	
Concrete	0.15	

Approximate values of specific weight for some rocks in Kg/m^3

Rock	Minimum value	Maximum value
Pumice	500	1100
Volcanic tuff	1100	1750

Rock	Minimum value	Maximum value
Tufaceous limestone	1120	2000
Coarse sand dry	1400	1500
Fine dry sand	1400	1600
Wet fine sand	1900	2000
Sandstone	1800	2700
Dry clay	2000	2250
Soft limestone	2000	2400
Travertine	2200	2500
Dolomite	2300	2850
Compact limestone	2400	2700
Trachyte	2400	2800
Porphyry	2450	2700
Gneiss	2500	2700
Serpentine	2500	2750
Granite	2550	2900
Marble	2700	2750
Syenite	2700	3000
Diorite	2750	3000
Basalt	2750	3100

10.1.1 Conversion Tables

Converting slope inclination into degrees and vice versa

Inclination (%)	Angle (°)		Inclination (%)	Angle (°)
1	0.5729		26	14.5742
2	1.1458		27	15.1096
3	1.7184		28	15.6422
4	2.2906		29	16.1722
5	2.8624		30	16.6992
6	3.4336		31	17.2234
7	4.0042		32	17.7447
8	4.5739		33	18.2629
9	5.1428		34	18.7780
10	5.7106		35	19.2900
11	6.2773		36	19.7989
12	6.8428		37	20.3045
13	7.4069		38	20.8068
14	7.9696		39	21.3058
15	8.5308		40	21.8014
16	9.0903		41	22.2936
17	9.6480		42	22.7824
18	10.2040		43	23.2677
19	10.7580		44	23.7495

20	11.3099		45	24.2277
21	11.8598		46	24.7024
22	12.4074		47	25.1735
23	12.9528		48	25.6410
24	13.4957		49	26.1049
25	14.0362		50	26.5651

Forces conversion

From	To	Operation	Factor
N	kg	Divide by	9.8
kN	kg	Multiply by	102
kN	Tone	Divide by	9.8
kg	N	Multiply by	9.8
kg	kN	Divide by	102
Tone	kN	Multiply by	9.8

1 Newton (N) = 1/9.81 Kg = 0.102 Kg 1 kN = 1000 N

Pressures conversion

From	To	Operation	Factor
Tons/m²	kg/cm²	Divide by	10
kg/m²	kg/cm²	Divide by	10000
Pa	kg/cm²	Divide by	98000
kPa	kg/cm²	Divide by	98
Mpa	kg/cm²	Multiply by	10.2
kPa	kg/m²	Multiply by	102
Mpa	kg/m²	Multiply by	102000

1 Pascal (Pa) = 1 Newton/mq 1 kPa = 1000 Pa

10.2 Shortcut commands

The bar shown in figure below can be used for a variety of functionalities:

1) With the shortcut letters of the menu followed by Enter you have quick access to commands.

Ex: N + Enter to create a new file.

2) You can ask a question followed by ? + Enter. In this case an advanced research will be made in the Help manual.

Ex.: Seism+?+Enter for information on seismic analysis.

3) Opening a program in a quick way.

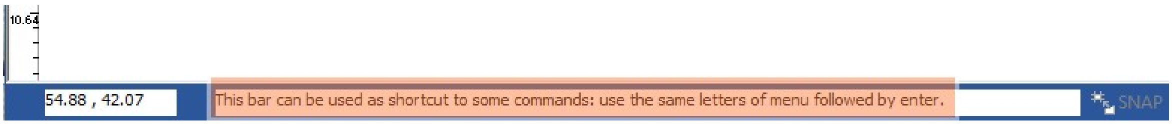
Ex.: MDC+Enter to open GeoStru MDC software.

4) Quick access to GeoStru contacts.

Ex.: Contact+?+Enter to access the contact list.

5) Quick access to web features:

Ex.: www.geostru.eu+Enter or info@geostru.eu+Enter



Shortcut commands bar

11 Contact

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