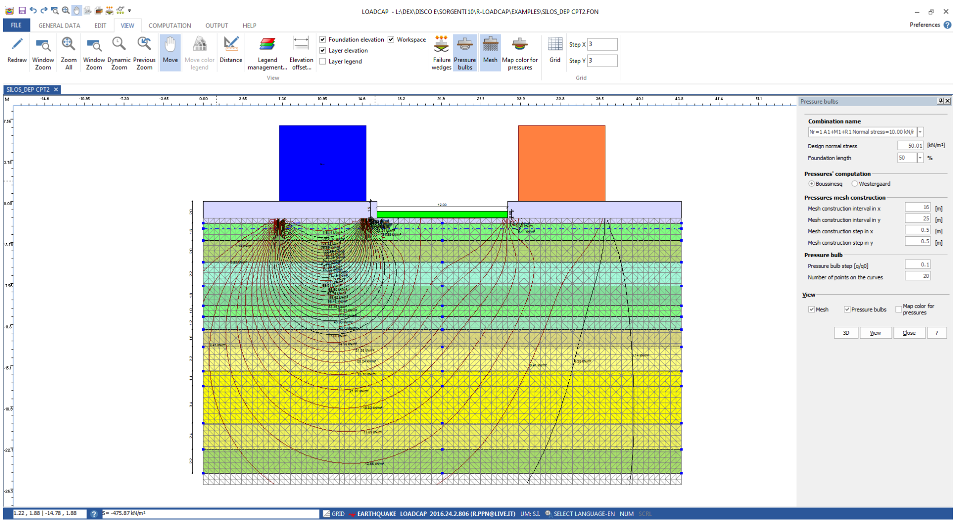
1 Loadcap

Loadcap is a software for the bearing capacity and settlements computation on rocks and loose soils, according to the methods of: Terzaghi (1955), Meyerhof (1963), Vesic (1975), Brinch Hansen (1970), Meyerhof and Hanna (1978), Richards et al. (1993) and computation of bearing capacity factors. In the software it is possible to chooce several types of settlements such as: Elastic (Timoshenko e Goodier, 1951); ii) Oedometric; iii) Schmertmann (1970) and Schmertmann et al. (1978), iv) Burland e Burbidge (1985); v) post-seismic (Boulanger e Idris, 2004; Idriss e Boulanger, 2008; Pradel, 1998; Yasuhara e Andersen, 1991)
Moreover, Loadcap solves liquefaction cases using the method of Seed and Idriss (1971) and allows to carry out soil reinforced analysis with geogrids by computing of the increase bearing capacity, deformation in the reinforcements, strain tensile force for membrane effect.



Bearing capacity, settlments – Loadcap: calculated at any point either inside or outside the foundation.

Bearing capacity in seismic conditions: **SHIKHIEV & JAKOVLEV, RICHARDS**

Diagrams: Bearing capacity as a function of the foundation base, of the depth of the bearing surface, of the acting loads. Unlimited number of layers. Display of pressure bulbs and failure wedges.

Supported computation standards:

- Eurocode 7/8
- British Codes BS8004

• Other standard…

DATA INPUT

- Multi level undo-redo
- Numeric input in tabular form
- Graphic input
- Automatic conversion of measurement units

GENERAL FEATURES

- Strip footing; Spread footing; Mat foundation; Circular foundation; Foundation on slope
- Bearing capacity according to: *Terzaghi, Meyerhof, Hansen, Brinch-Hansen, Vesic, Zienkiewicz, Eurocode, Meyerhof & Hanna*
- Settlements: *Elastic, Oedometric, Schmertmann, Burland & Burbidge* with progress over time
- Permanent settlements after the earthquake: Idriss and Boulanger, Pradel, Yasuhara and Andersen
- Seismic corrections: SHIKHIEV & JAKOVLEV
- Presence of Gorund Water Table
- Analysis in terms of total and effective stresses
- Display of pressure bulb and failure wedges 2D, 3D
- Computation of the stress state induced by external loads at any point
- Analysis in relation to total and effective tension
- Display of tensional state and pressure bulb
- Computation of stress state induced by external loads at any point
- Nspt correlation with geotechnical parameters according to: *Meyerhof, Sanglerat & Peck, Hanson and Thorburn*
- Verification to translation
- Analysis of imbedded plans
- Automatic reading of stratigraphic columns generated by Stratigrapher software
- Data exchange with Micropiles and Piles for foundations software
- Automatic computation of loads acting on foundation
- Computation of tensions: *Boussinesq,Westergaard*
- Interactive graphic construction
- Multi level undo-redo function
- Embankment analysis and settlement computation
- Differential settlements 3D

COMPUTATION OPTIONS

- Computation of the bearing capacity for multi layered soils using a weighted averages and punching verification
- Foundations on slopes
- Foundations with inclined bearing surface
- Foundations subject to eccentric loads
- Generating the bearing capacity table as a function of depth and width, exportable in Excel or in memory
- Construction of graphics allowable load-depth
- Construction of graphics allowable load-foundation width

- Load – settlement diagram
- Computation of subgrade reaction modulus using the model of Terzaghi and Bowles
- Water table, even above the bearing surface
- Display of pressure bulbs – Boussinesq or Westergaard
- Display of stress states in any point of the foundation soil
- Display of failure wedges
- Setup of influence area
- Computation of oedometric settlements in any point inside or outside the foundation

Note:

Geostru company created a service available on the Geoapp web page where there are several applications for making online calculations. Some of these can be used together with Rock Plane, for example: Bearing capacity; Lithostatic tensions; Liquefaction (Boulanger, 2014); etc., more details are shown in the Geoapp Section of this Help.

1.1 Project

1.1.1 New project

Creation of a new project

In order to start a new project for the calculation of the bearing capacity and settlements using LoadCap click on "File" menu and then “New”. From here a window “Project” will open from which it will be possible to enter the various data useful for the calculation (**Fig.A**).

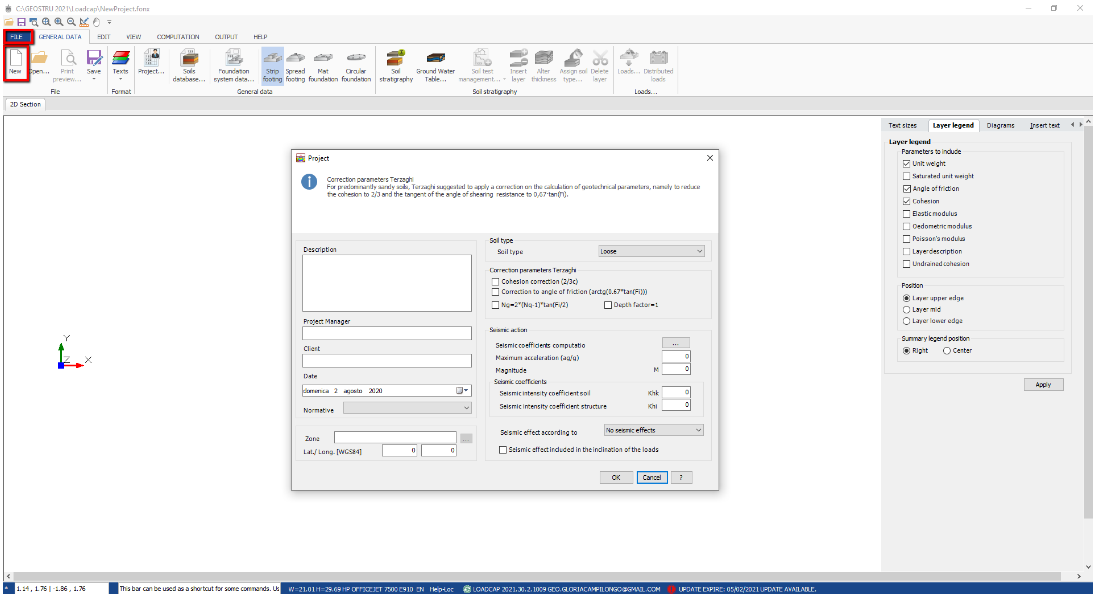


***Fig. A** – New project*

Once clicked “ok”, another window “Foundation system data…” will appear. In the cells of this window the data relating to the foundations and GWD “Ground Water Depth”, if present, must be entered. The numeric values already present are default data. Then clic on “apply” to save new values and then “ok”. These data can be entered and modified also by clicking on “general data” bar (**Fig.B**) and, based on the new data, the foundation will appear graphically.

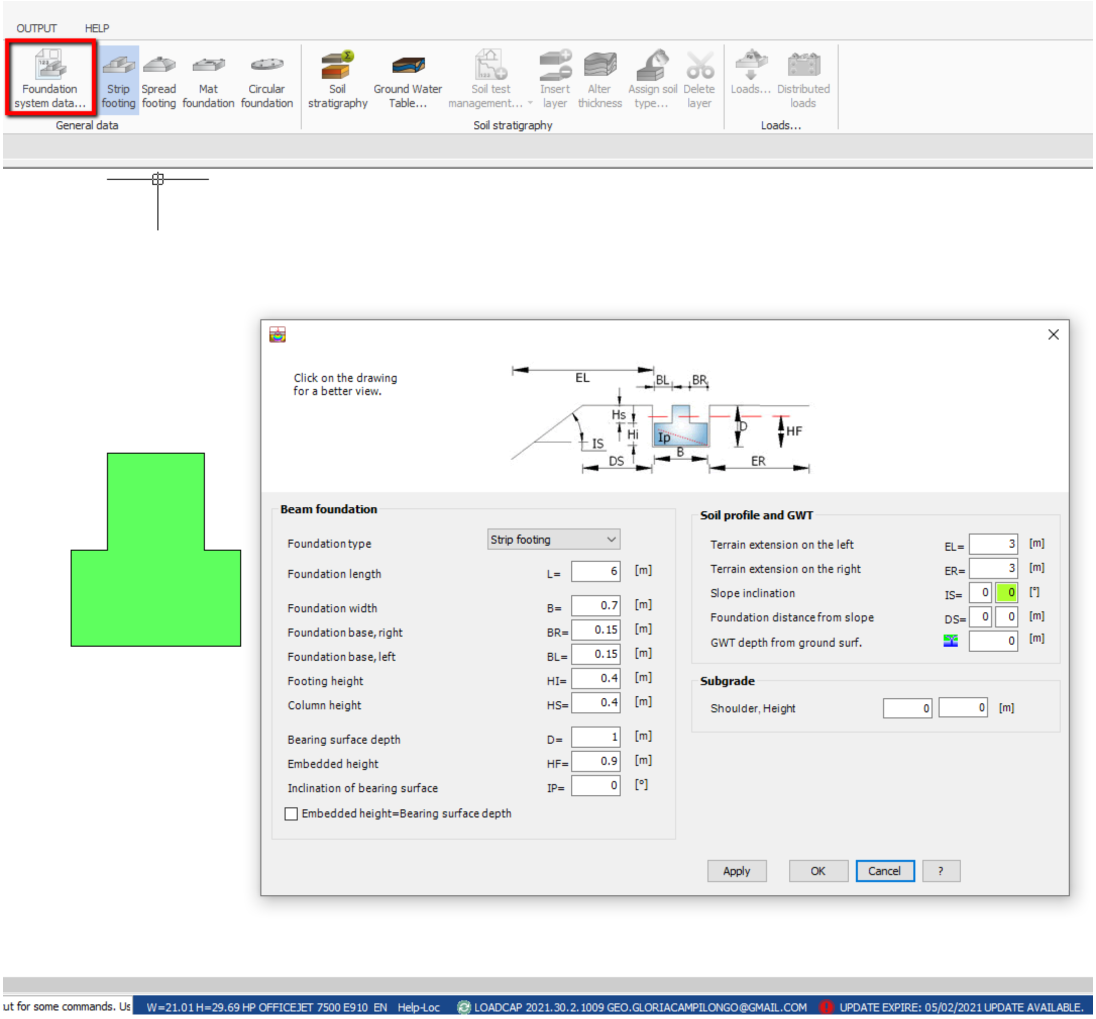


Fig.B *– Foundation data*

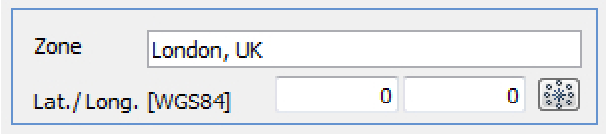
1.2 General data

General data menu allows the user to enter specific details of the current project:

- Description;
- Project Engineer;
- Customer;
- Date.

Zone

By entering the location in this format: street xxxx, city, state, country, the work area it will be automatically identified. Alternatively, assign coordinates in WGS84 system in decimal degrees.



The system requires an internet connection for the identification of the area.

Soil type

Choose from loose soil or rock according to the type of soil on which rests the foundation.For foundations on rock, the program automatically adjusts the data window of the stratigraphy (ex. RQD).

Correction parameters

For predominantly sandy soils, Terzaghi suggested to apply a correction on the geotechnical parameters, which is to reduce the cohesion to 2/3 and the tangent of the shearing resistance angle to 0,67·tan(j).

Comment
We recommend applying this correction only to DA1/1 and il DA2 (EC7).

Seismic action
To estimate the seismic effects on the site is better, at this point, to select the seismic normative and the computation methods to be used.

1.2.1 Soils database

Soils Database

This command enables the management of a database of soil types to be used as descriptors and quick selectors of soil layer attributes.
A dialog window opens that is composed of three columns, namely one for a textual name, one for geotechnical parameters and the last for association with a bitmap image to appear in the graphics.
A number of the more used soil types accompany this product from the producer, but these may be altered or expanded by the user at will using this tool.

New

To insert a new type, point to the word "Soils" in the left side column (the header word for the list) and open the floating menu associated with this (right click). Click on "New". The column in the center now appears with no data ready for input of the characteristics of the new type, which the user is invited to enter. Each lithology is identified in the list by the "Code" assigned by the user (central column).
To start entry click on the "New Type" text in the left side column. Then enter the characteristics required in the center column and select the bitmap texture that will characterize the soil in graphics from the third column.

Texture

To associate a bitmap drag the image from the left column to the box "Texture".
If alternatively only a color is required, click the "Texture" box and select a color from the resulting color palette.

Remove

To remove a type from the list (irrevocably!) select the type with the mouse and press "Delete" in the floating menu (right click).

1.2.2 Foundation system data

This command allows the definition of the geometrical data of the selected foundation type: strip footing, spread footing, mat foundation, circular foundation, in the presence of subgrade or ground water table. Colors can also be assigned in this window.

Strip footing
Considers the strip footing typology.

Spread footing
Considers the spread footing typology.

Mat foundation
Considers the mat foundation typology.

Circular foundation
Considers the circular foundation typology.

Foundation length (L)
Enter dimension in meters. Inactive for circular foundations.

Foundation width (B)
Enter dimension in meters. (corresponding to the diameter for circular foundation).

Footing base, left (BL)
Enter dimension in meters. Inactive for mat foundations or circular foundations.

Footing base, right (BR)
Enter dimension in meters. Inactive for mat foundations or circular foundations.

Footing height (HI)
Height of the lower side of the foundation in meters.

Column height (HS)
Height of the upper side of the foundation in meters. Inactive for mat foundations or circular foundations.

Bearing surface depth (D)
Represents the distance from the ground surface to the foundation base (bearing surface), in meters.

Embedded height (HF)
Gives the terrain height above the bearing surface that is considered in the bearing capacity term ($Nq \times \gamma \times D$).
The use of this option can be of value in cases where the bearing surface is some meters depth below ground level, in which cases the bearing capacity could become rather high.

Inclination of bearing surface (P)
Represents the inclination of the bearing surface of the foundation, positive if clockwise.

Selecting the option *"Embedded height = Bearing surface depth"* the software performs the calculation of the bearing capacity considering the depth of the bearing surface entered in the geometrical data of the foundation.
Otherwise, the program assigns to the variable D the value of the "Embedded height". In case of foundations fully or partially embedded, the excessive depth of the bearing surface can lead to high values of the bearing capacity due to the high value of the term ($\gamma \cdot D \cdot Nq$), therefore it can be useful to perform the computation with the embedded height, by clearing the option above and enter the actual embedded part of the foundation in the ground.

Subgrade Shoulder, Height
Indicate the shoulder of the sub-foundation and the height in meters. In this case it is also possible to assign a color to the structure from those on the right side of the window.

Terrain extension on the left (EL)
Insert the extension of the ground surface to the left.

Terrain extension on the right (ER)
Insert the extension of the ground surface to the right.

Slope inclination (IS)
Represents the inclination of the slope to the left and right side of the foundation, positive if clockwise.

Foundation distance from slope (DS)
Represents the distance in meters from the foundation to the slope, to the left and right sides of the foundation.

GWT depth from ground surface
Enter depth of ground water table from ground surface. Where these two coincide enter GWT depth as 1cm=0,01m.

By clicking on the drawing, the clicked symbol will be highlighted in the window for a simpler input.

1.2.3 Soil stratigraphy

This window enables the properties of terrain layers and a display texture to be specified or displayed for amendment. The window consists of two panes. The major one is a table of layer properties one row to each layer. The other is a rotating list of bitmaps to represent different soil types grouped by major type (cohesive, cohesionless, rocks and others). Instead of entering each layer data, previously defined layer descriptions may be recalled from the database. The table contains the following columns:

Nr.
Order number of the layer
N.B.: Layer data should be entered from the upper layer and progressively downwards.

DB
Drop down list containing the terrain types currently present on the database (Manufacturer supplied and user added). Clicking on one fills the row. The data thus entered may be altered individually if opportune.

Layer thickness
Layer thickness in meters.

G_k
Unit weight of the layer.

$G_{k\ saturated}$
Saturated unit weight of the layer.

For analysis performed in terms of total stresses in the absence of ground water must be entered the saturated unit weight.

Fi_k
Characteristic parameter of the angle of shearing resistance, in degrees. Where the GWT is present, insert the effective parameter for analysis in drained conditions or zero for undrained condition.

Cohesion (c_k)

Terrain cohesion. Where the GWT is present, insert the effective parameter for drained condition the total parameter for undrained condition.

Undrained cohesion (cu_k)

Insert the total parameter of the terrain cohesion for analysis in undrainded conditions.

Modulus of Elasticity

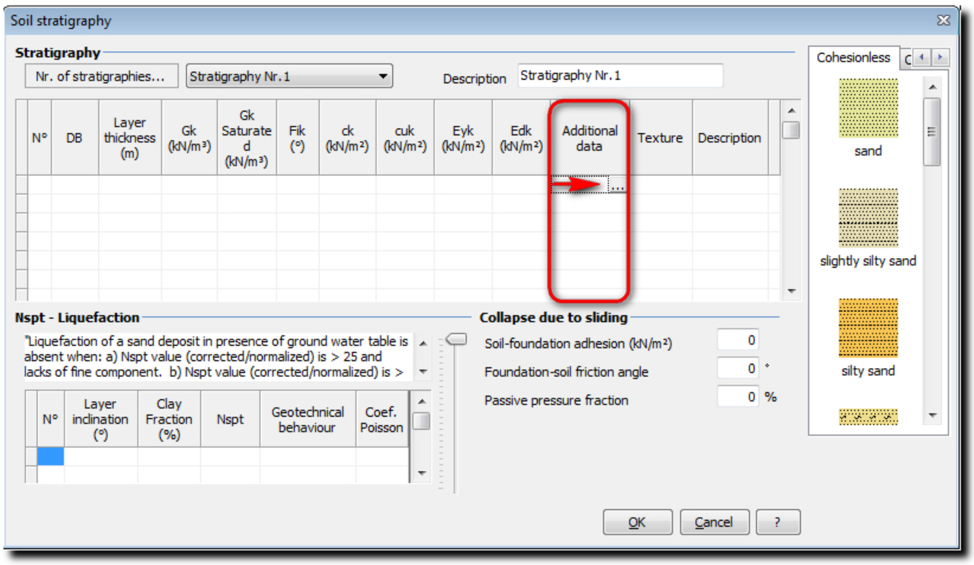
Elasticity or Young's modulus for the terrain. The parameter is required for Schmertmann computation of settlements. If this is not specified, settlements are calculated with the oedometric method (requires entry of oedometric modulus value). If both modules are specified then settlement is calculated by the oedometric method.

Oedometric Modulus

Deformation modulus obtained though the oedometric tests (conditions of inhibited lateral expansion). This parameter is required to evaluate the settlement by the oedometric method. If Young's modulus is assigned by the user instead of the oedometric modulus, the settlements are calculated by the method of Schmertmann.

Additional data

In order to calculate the oedometric settlements, consolidation settlements or post-seismic settlements additional data needs to be entered by activating the "Additional data" window as shown in the image below:

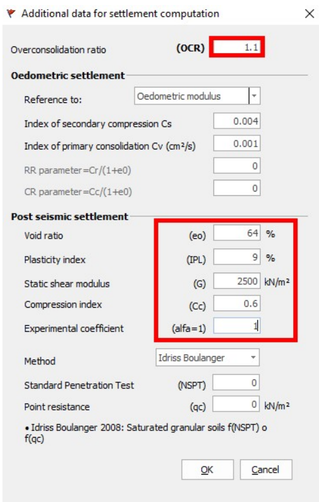


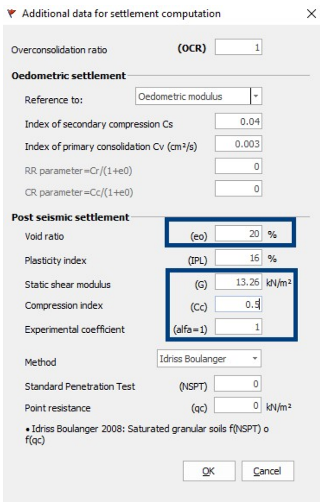
A subsidiary dialog window opens "Additional data for settlement computation" for the specification of the parameters needed for the computation of oedometric settlements and post-seismic settlements.

In the calculation of oedometric settlements, the user can choose to take into account the viscous effects or not from the drop down form: *"Oedometric modulus"* or *"Parameters RR, CR".*
In the first case it is necessary to define, for each layer, the oedometric modulus (E_d) and the coefficients C_s and C_v, where:

- C_s *(secondary compression index)*, parameter derived from the branch of a secondary compression of an oedometer test. Its value is necessary for the evaluation of secondary failure of a viscous nature.
- C_v *(primary vertical consolidation index)*, required for the computation of settlement over time with Terzaghi's mono dimensional method.
-

In the second case must be entered the RR and CR parameters (recompression and compression ratio) and have entered in the "Soil stratigraphy" window the value of the oedometric modulus of the layer. The settlement will be calculated without taking into account the secondary effects.
For the software to calculate the post-seismic settlements, for each layer, the user must declare a set of parameters as is highlighted in the images below:





Parameters necessary for the computation of post-seismic settlements, in red for cohesive soils and in blue for granular soils

Texture
Selects texture/color to be associated with current layer. Click the mouse to open a color palette from which the color to be applied can be chosen. Note that if the row was filled from the database, the color will probably already be assigned, but it can be altered in the same way. If otherwise it is desired to assign a texture, select a relevant one from the list in the right side pane and drag it to this column.

Description
In this cell the user can type a text for the description of the of the corresponding lithology.

If in "General data" was chosen "Rock" for "Soil type", in the table of the stratigraphy will be required, for each layer, the parameter **RQD** (Rock Quality Designation). Assign a value between 0 and 1.

Nspt - Liquefaction

In the presence of soils consisting of loose sands in presence of water table, even if they contain a fine silt-clay fraction, it should be verified the susceptibility to liquefaction using one of the methods generally adopted by the geotechnical engineering.

Clay Fraction %
Percentage of fine silt-clay fraction.

Nspt
Average number of blows in the layer obtained from a SPT soil test, can be dynamically assigned by going on the layer and moving the graduated cursor with the mouse.

The data included in the grid described above also apply to the computation of settlements with the Burland and Burbidge Method

Geotechnical behaviour
Indicate whether the layer is cohesive or cohesionless.

Layer inclination
Indicate the inclination of the layer.

Poisson's ratio
Poisson's ratio value for the layer. This parameter is required for computation of increments of tension below the foundation according to Westergaard.

Sliding verification - Collapse due to sliding

In accordance with the design criteria for ultimate limit state, the stability must be verified for collapse due to sliding besides the verification for general failure. In case of collapse due to sliding the resistance is calculated as the sum of a component due to adhesion and a component due to foundation-soil friction, the lateral resistance resulting from passive soil thrust can be brought into account according to a percentage indicated by the user.

Soil foundation adhesion
Enter the value of the adhesion in the indicated unit measure.

Soil foundation friction
Enter the value of the shearing resistance angle in degrees at the base of the footing.

Passive thrust ratio
Indicate the percentage of passive thrust to consider in the verification for collapse due to sliding.

By not entering this data in the "Collapse due to sliding" section, the software will automatically assume the geotechnical data of the layer where the foundation rests.

Attention

For the sliding verification the user must enter the vertical and horizontal actions from the "Loads" command.

1.2.4 Soil test management

LoadCap has an interface with other GeoStru Software programs Dynamic Probing, Static Probing, Stratigrapher, MP, etc. To import a stratigraphy constructed with one of this programs select this option from the tool bar or from "Graphic input" menu and click in the area below the foundation. An "Open file" window opens to select the file to be imported.

Geotechnical characteristics are also imported in the current measurement unit system.

1.2.5 Graphic input

Insert layer

Enables a layer to be inserted graphically. After selecting the command click on the worksheet where the layer boundary is to be drawn. A dialog window with the depth appears which can be confirmed to draw the layer or canceled. Needless to say the point clicked should be below ground level and is treated as upper or lower boundary depending on its position respect to the section base.

Alter thickness

The thickness of a layer may be altered with this command. When active clicking on a layer causes a dialog window with the current depth of the layer to appear. Alter the figure and press OK. The user may continue to do this until the command is deactivated by again clicking on the menu item.

Assign soil type

Enables the soil type for a layer to be selected interactively. Click on layer and a window with a list of soil types appears. Select one of them and exit the window to apply.

The soils in the list are part of the Soil database.

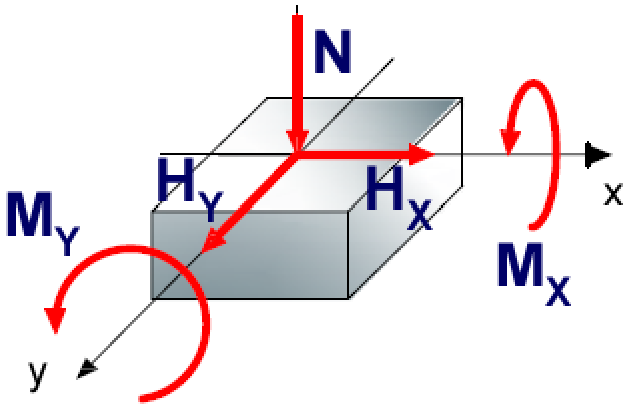
Delete layer

Enables a soil layer to be removed from the section. Click on the undesired layer and the layer is removed.

1.2.6 Loads

The loads acting on the foundation can be distinguished in design loads and operating loads.

The design loads are used for the computation of the bearing capacity. The values to be inserted are design values so they must be assigned, when an ultimate limit state



Sign convention

verification is carried out, together with the amplifying coefficients of the actions.

The operating loads are used for the settlement computation.
More design loads and operating loads combinations can be assigned simultaneously in order to perform the computation according to the normative.
Alternatively to loads can be assigned the design normal pressure.

The commands "Generate combinations" and "Assign loads" activate the number and the type of combination to adopt based on the normative chosen and an orientation value for the design normal pressure, if this data is not available.

Attention:

LoadCap does not calculate the weight of the foundation.

Automatic computation of pressures in the soil
Once inserted the components N, Mx, My, the software automatically calculates the pressure transmitted from the foundation to the soil.

1.2.7 Distributed loads

These are additional loads which can be assigned to the right or left sides of the foundation in order to take into account the presence of overloads adjoining the foundations (ex. bordering buildings). Their effect is only considered as an increase in the subsurface strain for the assessment of the settlements and in the interference of the bulbs.

1.3 Bearing capacity

To calculate the bearing capacity of the foundation various methods can be used:

- **Hansen's method**
  Select this method to determine bearing capacity on loose soils.

- **Terzaghi's method**
  Select this method to determine bearing capacity on loose soils.

- **Meyerhof's method**
  Select this method to determine bearing capacity on loose soils.

- **Vesic's method**
  Select this method to determine bearing capacity on loose soils.

- **EC-8 Method**

Choosing this option allows the computation of the bearing capacity according to the guidelines of Eurocode 7 (on geotechnical design) and Eurocode 8 (on earthquake).

- **Terzaghi's method on rock**
  Select this method to determine bearing capacity on rock.

- **Zienkiewicz's method on rock**
  Select this method to determine bearing capacity on rock.

Bearing capacity
The vertical and horizontal bearing capacity are calculated for each design combination.

Through the "Analysis options" command in the "Bearing capacity" computation window the user can select the type of analysis to be performed:

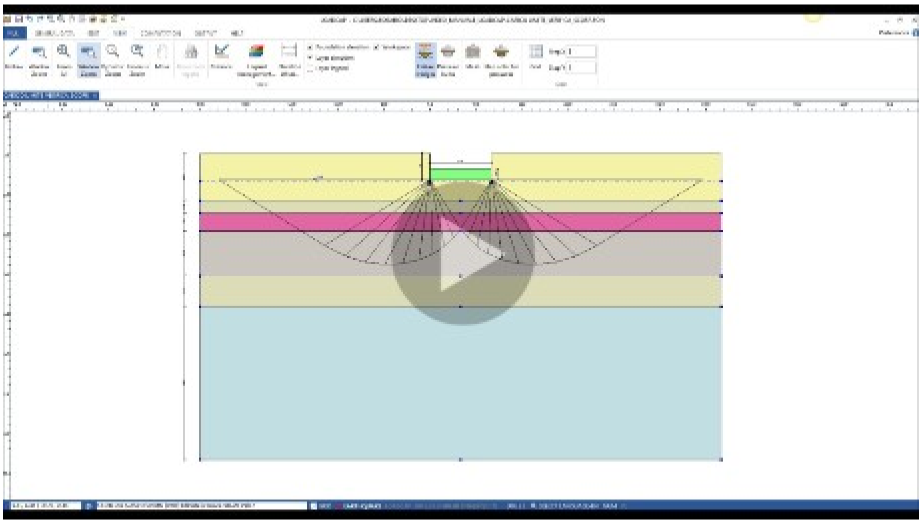
- Drained condition: Select this option to calculate the foundation bearing capacity in drained conditions (effective parameters).
- Undrained condition: Select this option to estimate the bearing capacity in undrained conditions (total parameters).
- Computation according to layers' weighted average: Select this option to calculate the foundation bearing capacity, based on the weighted average of the individual layer parameters. Not selecting this option causes only the parameters of the layer upon which the foundation is resting to be considered (Classic method).

For each combination the user can reduce the characteristic parameters of the soil according to the imposed reduction coefficients and make the Seismic corrections in accordance with the selected design approaches. For the calculation of seismic effects on the bearing capacity are proposed the maximum seismic ground accelerations that can be entered by the user.

> LoadCap automatically identifies the worst computation combination and marks it with the symbol *.

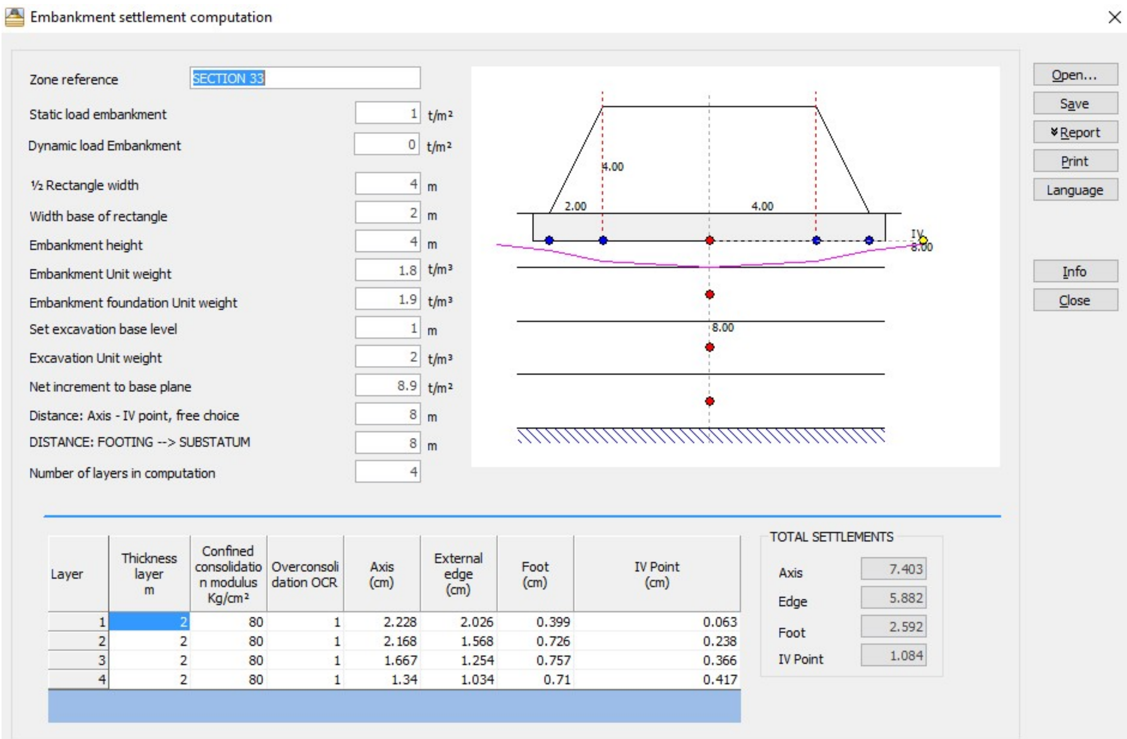
Ks Computation
The computation of this constant is performed using the method suggested by Bowles.

Sliding verification - Collapse due to sliding
See Soil stratigraphy



1.3.1 Embankment module

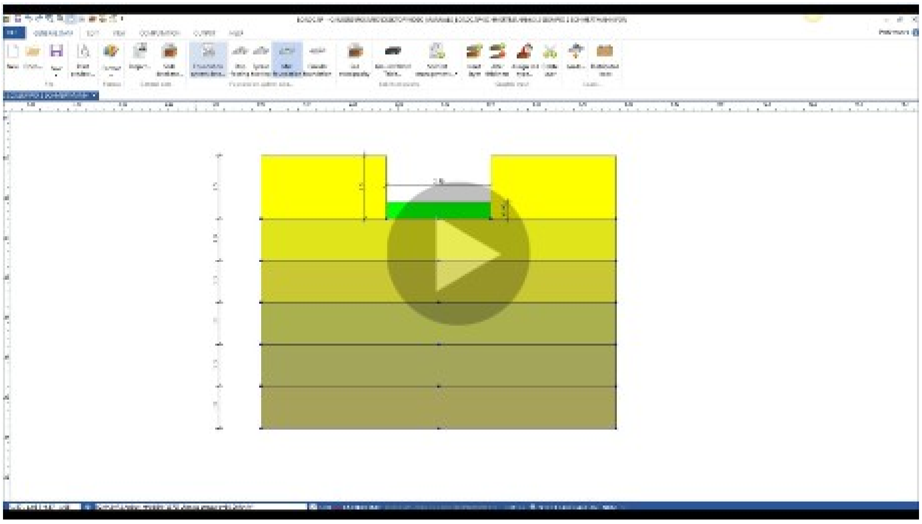
Using this module can be calculated the settlement of embankments with the oedometric method.
Pressure imposed on embankment: loads acting on the embankment (roads, etc.)
Set excavation base level: depth of the foundation. Net increment to base plan will be automatically calculated by the program.
Distance: Axis - IV point, free choice: the settlements are calculated in Axis, Center, Foot and a point chosen by the user, the IV point, for which is entered the "Distance axis IV point".
As input data of the stratigraphy must be assigned: the oedometric modulus and the overconsolidation ratio.



1.4 Settlements

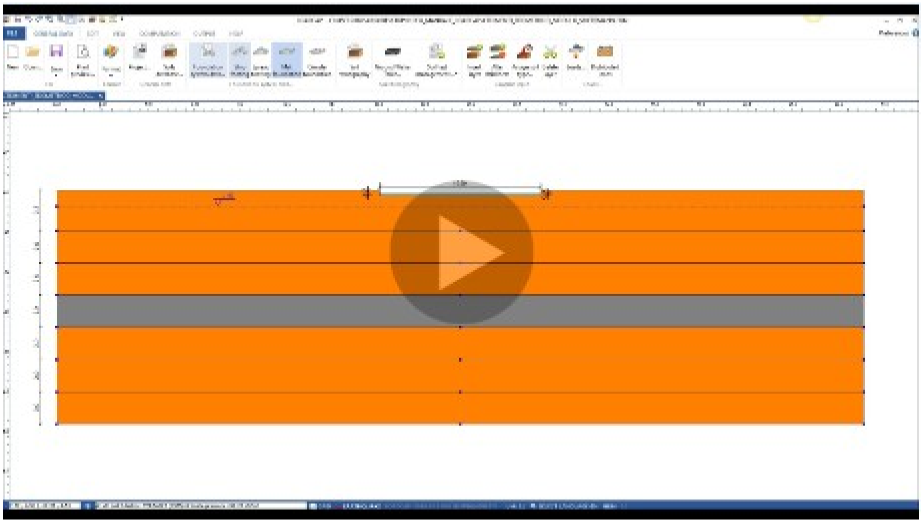
1.4.1 Oedometric settlements – Schmertmann

LoadCap performs the computation of settlements utilizing two approaches: oedometric and Schmertmann.
The program automatically selects the first approach when the user enters the oedometric modulus of the layers ("Soil stratigraphy" menu) and, for secondary settlement computation, the index of secondary compression (C_s). The second approach is selected when the above data is absent but the Elasticity modulus ("Soil stratigraphy" menu) is given in the same window.
When the results are displayed is shown the plan view of the foundation, a red dotted line highlights - the center line and the application point of the load (red point). Keep left click pressed while moving on the influence area to see total settlement. The value of the settlement is also shown in the blue line of the active table. The total settlement over time and the percentage of the settlement at t days after the application of the load is shown, layer by layer, in the table.



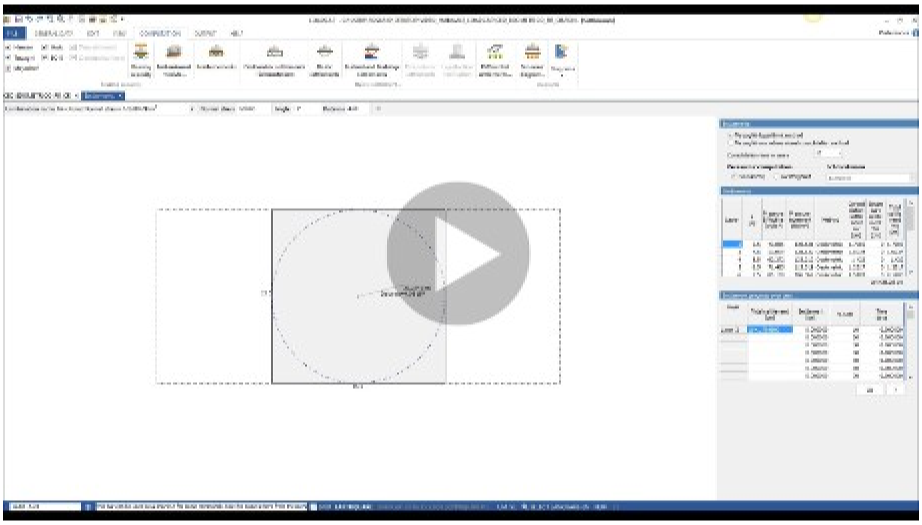
Observation

For a correct assessment of the settlement, the compressible layers must have a reduced thickness (<2.00 m), therefore, if there are soils with higher thickness is preferable that the user performs a subdivision of those layers in smaller layers, keeping the geotechnical characteristics of the origin layer and changing only the number of layers and their thickness.



See also

Soil stratigraphy, Elastic settlements, Burland & Burbidge settlements



1.4.2 Elastic settlements

The computation of elastic settlements (immediate) to the edge and center of the foundation are calculated using an equation based on the elasticity theory of Timoshenko and Goodier (1951).

1.4.3 Burland & Burbidge settlements

Computation of settlements using the method of Burland and Burbidge (coarse-grained soils), with various correction factors. For each layer is shown the average value of NSPT defined in the "Soil stratigraphy" window; this value can be modified.

1.4.4 Post-seismic settlements

It is calculated the post-seismic settlement of cohesive and granular liquefiable soils. The details of the computation are shown in the Post-Seismic Settlements.

Attention

It is necessary to enter, prior to calculation, the "Additional data" for the computation of the post-seismic settlements in the "Soil stratigraphy"window.

1.4.5 Liquefaction check

For each layer is viewed the liquefaction check with the method proposed by the CNR (Italian National Research Center) and suggested by GNDT (Italian National Group for the Defence against Earthquakes). The verification is performed only in the presence of seismic acceleration, cohesionless soils under GWT. The method CNR-GNDT – Tokimatsu and Yoshimi (1983).

The liquefaction resistance has the following expression

$$R = 0.26 \cdot \left[0.16 \cdot \sqrt{N_a} + \left(0.21 \cdot \sqrt{N_a}\right)^4\right]$$

With

$$N_a = \left(\frac{1.7}{\sigma_v + 0.7}\right) + N_1$$

$$\sigma_v \left(kg / cm^2\right) = \text{effective vertical stress}$$

$$N_1 = 0 \text{ for a fine percentage } p_c < 5\%, 10\, p_c + 4 \text{ for } p_c \geq 5\%$$

For the theoretical notes on this subject see the theoretical notes of this manual
See Soil stratigraphy.

1.4.6 Differential settlements

In this window can be calculated the differential settlements by assigning the "Stratigraphic areas" and "Load areas".

To each Stratigraphic area can be associated its own stratigraphy whose characteristics are assigned in the "Soil stratigraphy" menu, in "General data" main tab.

In the "Areas" section of the right side menu the user can, first of all, create the **"Stratigraphic areas"**.
For each of these area is assigned a "Description", the location in the plan through the coordinates "x", "y" and "z", the base "B" and length "L", a "Color" and its own "Stratigraphy".

For the **"Load areas"** are assigned the "Description", the position, the base "B", the length "L", the height "H" and the type "T", the "Color" and the "Load ".
The "T" option allows you to specify the use of a cylindrical load by assigning the character "c" or a rectangular load by entering "1".
For the correct input of data is necessary to use the ";" as a separator in the input box according to the standards suggested in the table header.

In the **"Preferences"** section of the side menu can be assigned parameters to be used for the analysis and synthesis of data.
Here can be modified the construction step of the mesh, both along the x abscissa and the y coordinate, the density of the settlement isobars and also the size of the texts.

The user can also choose to display the value of the settlement and the settlement isobars. The scale factor of textures and settlements allow better visualization of the results.

In the **"Analysis"** section of the side menu are generated the results that are reproduced in the diagram along with the mesh used. Scrolling with the mouse on the drawing shows the value of the settlement corresponding to the pointer.
After the analysis is done can be created a section corresponding to the red dotted line that can be adapted to suit the user's needs by using the mouse or by setting the values in the menu.

The **"View table"** option displays the values of settlements corresponding to distances for the calculated section.
This section, together with the table, can be dragged with the mouse in the drawing.

From the top menu can be chosen a solid 3D view of the work by using the **"Rendering"** command.
The **"Rotate"** command allows the rotation of the drawing in space, while the **"3D Wire"** command allows a vision of the project boundaries, settlements and the 3D mesh.
This option "2D" brings back to the plan view.

Importing external files

To perform the analysis of differential settlements on files imported from other programs file as shown below must be prepared:

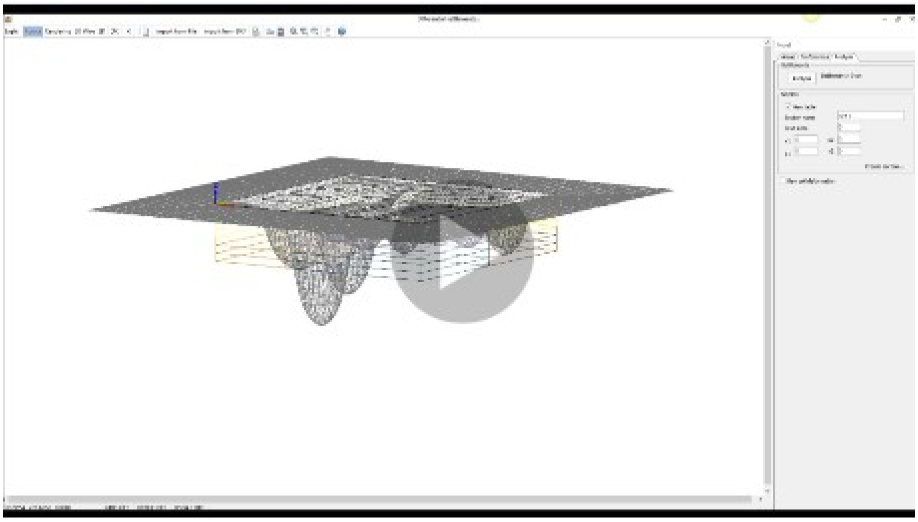
Descr	x	y	z	B	L	H	T	V
L1	2	2	0	2	2	0.5	0	100
L2	8	2	0	2	2	0.5	0	120
L3	14	2	0	2	2	0.5	0	130
L4	2	8	0	2	2	0.5	0	100
L5	8	8	0	2	2	0.5	0	120
L6	14	8	0	2	2	0.5	0	130
L7	2	14	0	2	2	0.5	0	100
L8	8	14	0	2	2	0.5	0	120
L9	14	14	0	2	2	0.5	0	130

The created file should be saved in TXT or CSV format, with values separated by tabs. Examples of import files are located in the program installation folder.

To import the file right-click on data grids.

Import from DXF file

The dxf file must be formed by rectangles which have to be drawn on the following layers: LAY1 for stratigraphic areas, LAY2 loading area. An example can be found in the example folder of the software.



1.5 Diagrams

Graphical parameters

The command gives the opportunity to modify the parameters of the graphics in the results report (bearing capacity - foundation base, bearing capacity - bearing surface depth), to choose the author for the computation of stresses in the subsoil, alter the length of the foundation in the table diagram report.

Stresses diagram

Displays a diagram showing the development of stresses in the subsoil, estimated at the center of the foundation, as a function of the depth z using the theory of Boussinesq or Westergaard (in base of the choice made in "Pressure bulbs")

Example: modifying the value of the bearing surface depth's step will modify the representation scale along the x axis of the (Q, D) diagram.

Diagrams report

Displays a table that shows the various values of the allowable load as a function on the bearing surface depth D and width B, for each length L chosen from among those proposed. The table is constructed based on the general settings of the graphics assigned in the menu.

Bearing capacity/depth diagram (Q D)

Displays a diagram showing the development of the bearing capacity (calculated with the chosen methods) as a function of the bearing surface depth D. For each pair of values of the base B and length L is constructed a diagram that can be copied (using "Copy" command from the "Edit" menu) or printed (from the menu "Output").

Bearing capacity/base diagram (Q B)

Displays a diagram showing the development of the bearing capacity (calculated with the chosen methods) as a function of the foundation length B. For each pair of values of the bearing surface D and the length L is constructed a diagram that can be copied (using "Copy" command from the "Edit" menu) or printed (from the menu "Output").

1.6 View

Legend management

Allows the customization of the layer legend (parameters of the layer to include, position)

Failure wedges

Displays the active, passive and transition failure zones in the work area.

Pressure bulbs

Displays pressure bulbs on the worksheet, that is the development with the depth of the ratio q/q0 where is the pressure induced by the load q0 applied on the bearing surface. Stress increase below the foundation may be calculated with Boussineq or Westergaard methods.

Pressure bulbs construction

Design normal pressure
Design normal pressure for the computation of stresses.

Foundation length
Corresponds to the foundation section for which the bulb is represented. For example if entered 50% the bulb will be drawn at the middle and then the pressure values will be referred to the middle section of the foundation.

Mesh construction interval in x and step
Amplitude, along the x axis, of the meshing needed to build the pressure bulb. The step represents the amplitude of the cells.

Mesh construction interval in y
Amplitude, along the x axis, of the meshing needed to build the pressure bulb.

Note:

It may occur that, as shown in the image A below, the bulbs are only partially visible.
To view them completely or in a broader way, a calibration must be done based on the size of the foundation and the values predefined of the interval and the construction step of the mesh in both x and y directions, as shown in the image B.

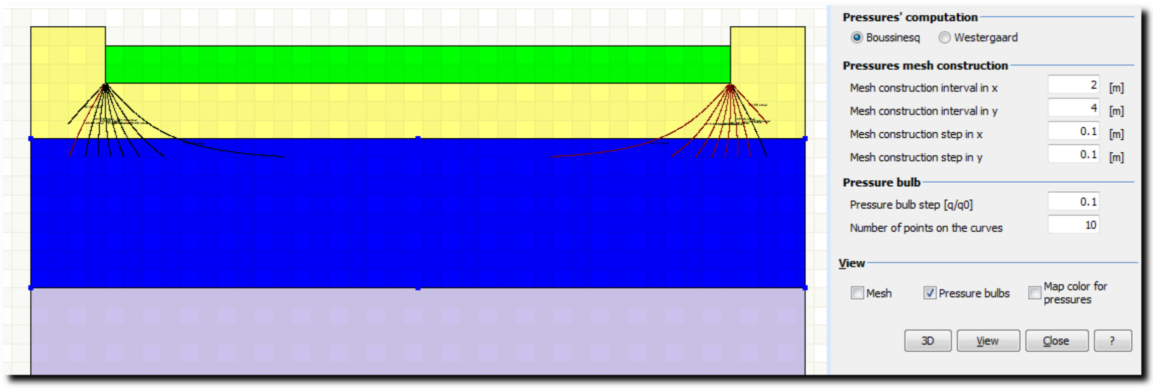


Image A- Partial view of the pressure bulbs

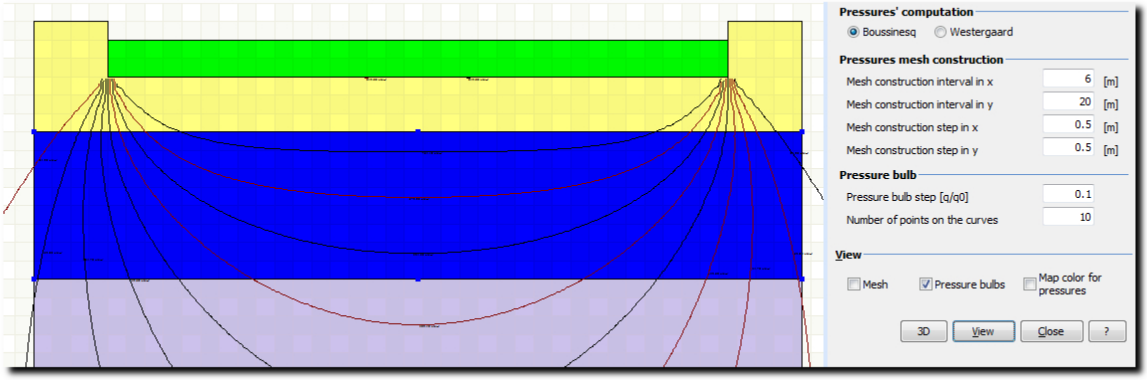


Image B- Calibrated view of the pressure bulbs

Mesh
Displays on the worksheet, the triangular mesh used for the modeling of the progress of the stress state below the bearing surface.

Map color for pressures
Displays pressure bulbs on the worksheet as colored areas.

1.7 Output

Options

Offers the possibility to assign textures, the graphics of the work area, setup the text report (margins of the pages, tables, choose whether to include the theoretical note in the report or not), choose the units of measurement system (S.I. or Technical) fill the company data and set the saving options.

Create report

Creates a text report of the computation performed with the software. The settings of the report can be assigned/modified using the "Options" command.

Export in DXF

Creates a drawing of the work area (foundation, layers, legends, elevations, etc.).

Export BitMap

Saves the image of the work area.

Export to GFAS and Export to Slope

Prepares the files for the export in GFAS software (finite element software for the soil mechanics) and Slope software (software for slope stability of loose soil or rock slopes), both developed by GeoStru Software.

1.8 Geoapp

Geoapp: the largest web suite for online calculations

The applications present in Geostru Geoapp were created to support the worker for the solution of multiple professional cases.
Geoapp includes over 40 applications for: Engineering, Geology, Geophysics, Hydrology and Hydraulics.

Most of the applications are **free**, others require a monthly or annual **subscription**.

Having a subscription means:

- access to the apps from everywhere and every device;
- saving files in cloud and locally;
- reopening files for further elaborations;
- generating prints and graphics;
- notifications about new apps and their inclusion in your subscription;
- access to the newest versions and features;
- support service throught Tickets.

1.8.1 Geoapp Section

General and Engineering, Geotechnics and Geology

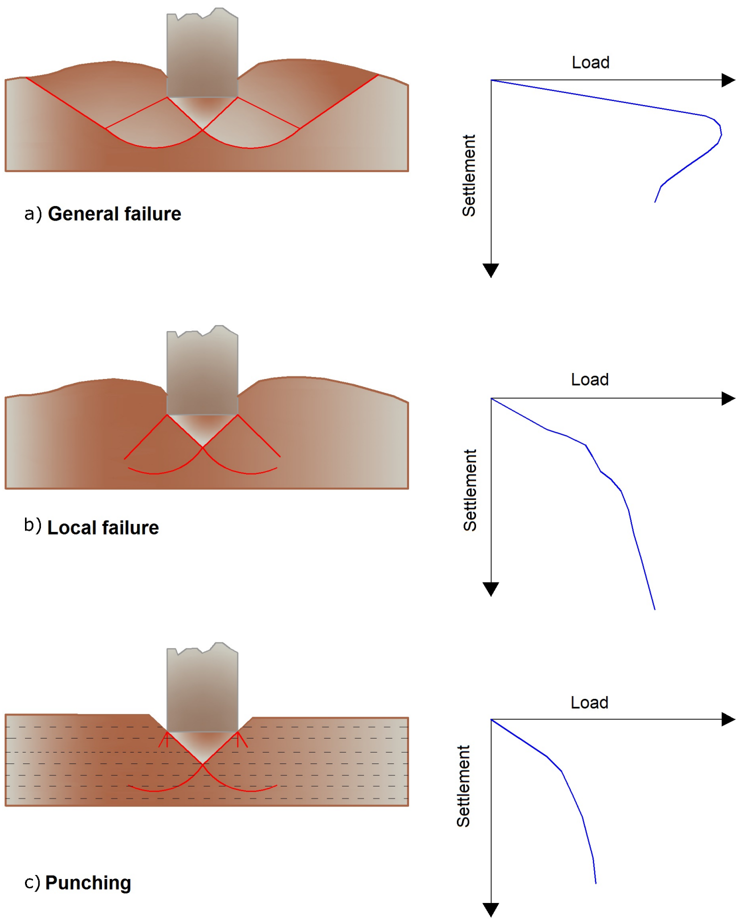
Among the applications present, a wide range can be used for Loadcap. For this purpose, the following applications are recommended:

- Bearing capacity
- Lithostatic tensions
- Foundation piles, horizontal reaction coefficient
- Liquefaction (Boulanger 2014)
- Reinforced lands

1.9 Theoretical notes

The bearing capacity of a shallow foundation can be defined as the maximum value of the load applied, for which no point of the subsoil

reaches failure point *(Frolich method)* or else for which failure extends to a considerable volume of soil *(Prandtl method and successive).*
Experimental observations have shown that the soil can get to failure through three mechanisms (see image below): General failure is characterized by the formation of well-defined sliding surfaces that start from the foundation and reach the ground level, and a swelling of the soil at the sides of the foundation. Punching failure when the lowering of the foundation is made possible by the formation of vertical shear planes along the perimeter, without generating sliding surfaces. Local failure corresponds to the formation of a clear sliding surface below the foundation, which is dispersed in the adjacent soil, it shows a tendency to swelling timid side of the ground.



Types of soil failure

The solutions available for the calculation of the bearing capacity are based on the assumption of rigid-plastic behavior of the soil and are therefore, strictly speaking, only applicable to the case of general failure. It can be shown that the bearing capacity of a soil is the sum of three factors: soil weight γ', q' overload and cohesion c'; solutions available today have been obtained by the superposition of individual independent problems.

Prandtl (1921) has studied the problem of failure of an elastic half-space due to a load applied on its surface with reference to steel, characterizing the resistance to failure with a law of the type:

$$\tau = c' + \sigma' \cdot tg\varphi' \qquad (1\text{-}1)$$

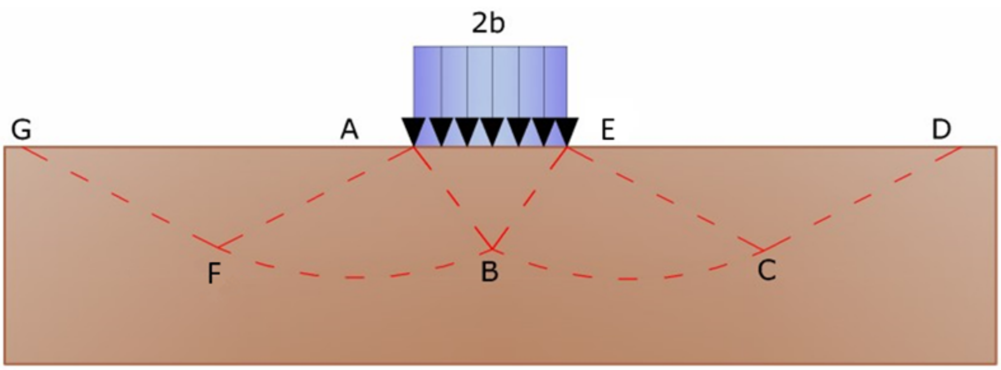
valid even for soils.

Prandtl assumes:

- Rigid- plastic behaviour
- Failure resistance of the material expressed with the relationship (1-1)
- Uniform vertical load applied to an infinitely long strip of width 2b (Plane strain case)
- No tangential shear on the interface between load strip and boundary surface of the half-space
- No overload the edges of the foundation (q'=0)

In the space between the upper (where the load lies) and lower (indicated by GFBCD) surface, simultaneously with failure, the plasticization phenomena occurs.

Within the triangle AEB failure occurs according to two families of straight segments inclined by 45°+φ/2 to the horizontal.
Within zones ABF and EBC failure occurs along two families of lines, the ones made up of straight lines passing through points A and E, and the other consisting of arcs of logarithmic spirals. The poles of these are points A and E. In the triangles AFG and ECD failure occurs along segments inclined at ±(45°+φ/2) to the vertical.



Solution of Prandtl

Once the soil tending to failure by application of the bearing capacity is identified, it can be calculated expressing the equilibrium between the forces acting in any volume of soil whose base is delimited by whichever slip surface.

Thus we reach the equation:

$$q_{\lim} = c^{'} \cdot B$$

where the coefficient B depends only upon the soil’s angle of friction φ'. For$\varphi'\neq 0$ the factor B = 5,14.

In the alternate case, namely that where the soil is cohesionless (c=0, $\gamma'\neq 0$) q'=0 so that according to Prandtl it would not be possible to apply any load to cohesionless soils.

This theory, even if not practically applicable, has been the base for all the following researches and computation methods.

Caquot, indeed, continued from the same premises as Prandtl excepting that the load strip is no longer placed on the surface but at a depth h, where h < 2b; the soil between surface and the depth h has the following characteristics: $\gamma'=0$, $\varphi'\neq 0$, c'=0 and is a material with weight attribute but with no resistance.

Solving the equilibrium equations it is possible to obtain the following expression:

$$q_{\lim} = A \cdot \gamma' + B \cdot c'$$

which is certainly a step forward but hardly reflects reality.

Terzaghi (1955)

Terzaghi, continues on the same lines as Caquot but adds modifications to take into account of the real characteristics of the foundation-soil system. Under the action of the load transmitted by the foundation, the soil at the contact with the foundation tends to move laterally, but is restrained in this by the tangential resistances that develop between the soil and the foundation. This results in a change of the stress state in the ground placed directly below the foundation.
Terzaghi assigns to the sides AB and EB of Prandtl’s wedge, an inclination Ψ to the horizontal, assigning to this a value as a function of the mechanical characteristics of the soil at the contact soil-foundation.
Thus $\gamma'=0$ for soil below the foundation is reviewed assuming that the failure surfaces remain unaltered, the expression for bearing capacity becomes:

where:

C is a coefficient that is a function of the angle of friction φ of the soil below the footing and of the angle φ defined above
b is the half width of the strip

Furthermore, on the basis of experimental data, Terzaghi introduces factors due to the foundation shape.
Again Terzaghi refines the original hypothesis of Prandtl (who considered the behaviour of soil as rigid–plastic) and assigns such behaviour only to very compact soils. In these soils the loads/settlements curve is linear at firstand then becomes short curved (elastic-plastic behavior). Failure is instantaneous and the value of the bearing capacity is easily identifiable (general failure).
In a very loose soil however the relation loads/settlements has an accentuated curved line even at low levels of load due to a progressive failure of the soil (local failure) and thus the identification of bearing capacity is not so clear like for compact soils..
For very loose soils Terzaghi considers the previous formula introducing some reduced values of the soil mechanical properties:

$$\mathrm{tg}\varphi'_{rid} = 2/3 \cdot \mathrm{tg}\varphi'$$
$$c'_{rid} = 2/3 \cdot c'$$

Thus Terzaghi’s formula becomes:

$$q_{ult} = c \cdot N_c \cdot s_c + \gamma \cdot D \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \cdot s_\gamma$$

where:

$$N_q = \frac{a^2}{2\cos^2(45 + \varphi'/2)}$$

$$a = e^{(0.75\pi - \varphi'/2)\tan\varphi'}$$

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

$$N_\gamma = \frac{\tan\varphi'}{2}\left(\frac{k_{p\gamma}}{\cos^2\varphi'} - 1\right)$$

Foundation type:	Strip	Circular	Square
S_c	1.0	1.3	1.3
S_q	1.0	0.6	0.8

Meyerhof (1963)

Meyerhof proposed a formula for bearing-capacity calculation similar to that of Terzaghi but introduced more foundation shape factors as sq that multiplies with the depth term Nq; including furthermore, depth factors di

and inclination factors ii for the cases where the load line is inclined to the vertical.
Meyerhof obtained the N factors by making trials with BD arc (see Prandtl mechanism) which include an approximation for shear along "shallow" foundations (soil lateral support)..
The N factors are given below together with the complete formula:

Vertical load $$q_{ult} = c' \times N_{c'} \times s_{c'} \times d_{c'} + \gamma' \times D \times N_{q'} \times s_{q'} \times d_{q'} + 0.5 \times \gamma' \times l$$

Inclined load $$q_{ult} = c' \times N_{c'} \times i_{c'} \times d_{c'} + \gamma' \times D \times N_{q'} \times i_{q'} \times d_{q'} + 0.5 \times \gamma' \times l$$

$$N_q = e^{\pi \tan \varphi} \tan^2(45 + \varphi/2)$$

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

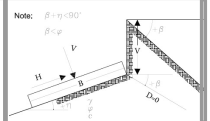
$$N_\gamma = (N_q - 1) \tan(1.4\varphi)$$

Factors of	Value	For
Shape	$s_c = 1 + 0.2 \times k_p \frac{B}{L}$	whatever j
	$s_q = s_\gamma = 1 + 0.1 \times k_p \frac{B}{L}$	φ >10
	$s_q = s_\gamma = 1$	φ =0
Depth	$d_c = 1 + 0.2\sqrt{k_p}\frac{D}{B}$	whatever j
	$d_q = d_\gamma = 1 + 0.1\sqrt{k_p}\frac{D}{B}$	φ >10
	$d_q = d_\gamma = 1$	φ =0
Inclination where : $k_p = \tan^2\left(45^\circ + \frac{\varphi}{2}\right)$ θ=inclination of the resultant on the vertical	$i_c = i_\gamma = \left(1 - \frac{\vartheta}{90}\right)^2$	whatever j
	$i_\gamma = \left(1 - \frac{\vartheta}{\varphi}\right)^2$	φ >10
	$i_\gamma = 0$	φ =0

Factors of form, depth and inclination that appear in the formula of Meyerhof

Hansen (1970)

Hansen's formula is considered like a further extension of the earlier Meyerhof equation. These extensions consist of the introduction of bi that considers the possible tilting of the footing from the horizontal and a ground factor gi for the possibility of a slope of the ground supporting the footing.
Hansen's formula is valid for whatever ratio D/B and therefore for both shallow footings and deep bases, however the author introduces coefficients to compensate for the excessive increment in bearing capacity with the depth increasing.

<table>
<tr><th>Shape factors</th><th>Depth factors</th><th>Load inclination factors</th><th>Soil inclination factors</th><th>Bearing surface inclination factors</th></tr>
<tr><td>$s'_c = 0.2\frac{B}{L}$</td><td>$d'_c = 1 + 0.4k$</td><td>$i'_c = 0.5 - 0.5\sqrt{1 - \frac{H}{A_f c_a}}$</td><td>$g'_c = \frac{}{1}$</td><td>$b'_c = \frac{}{1}$</td></tr>
<tr><td>$s_c = 1 + \frac{N_q}{N_c}\frac{B}{L}$</td><td>$d_c = 1 + 0.4k$</td><td>$i_c = i_q - \frac{1 - i_q}{N_q - 1}$</td><td>$g_c = 1$</td><td>$b_c = 1$</td></tr>
<tr><td>$s^{***}_c = 1$</td><td>$d_q = 1 + 2\tan\varphi(1 - \sin\varphi$</td><td>$i_q = \left(1 - \frac{0.5H}{V + A_f c_a \cot\varphi}\right)^5$</td><td>$g_q = g$</td><td>$b_q = ex$</td></tr>
<tr><td>$s_q = 1 + \frac{B}{L}\tan\varphi$</td><td>$d_\gamma = 1 \forall \quad \phi$</td><td>$i^*_\gamma = \left(1 - \frac{0.7H}{V + A_f c_a \cot\varphi}\right)^5$</td><td colspan="2" rowspan="3">
• the expressions with (') are valid when φ=0.
• A_f= effective area of the foundation (B' x L')
• D= depth of the foundation in the soil, to be used with B and NOT with B'.
• c_a= the adhesion to the base, equal to the cohesion or to a fraction of it</td></tr>
<tr><td>$s_\gamma = 1 - 0.4\frac{B}{L}$</td><td></td><td>$i^{**}_\gamma = \left(1 - \frac{(0.7 - \eta/450)H}{V + A_f c_a \cot\varphi}\right)^5$</td></tr>
<tr><td colspan="3">$$k = \frac{D}{B} \quad if \quad \frac{D}{B} \le 1$$ $$k = \tan^{-1}\frac{D}{B}(rad) \quad if \quad \frac{D}{B} > 1$$
* η=0
** η>0
*** strip foundations</td></tr>
</table>

Factors proposed by Hansen for the computation of q_{ult}

D/B	0	1	1.1	2	5	10	20	100
d'c	0	0.40	0.33	0.44	0.55	0.59	0.61	0.62

Vesic (1975)

Vesic proposes a formula that is analogous to taht of Hansen's with Nq and Nc Meyerhof's terms and Nγ as follows:

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

Shape and depth factors are the same than Hansen’s but there are differences in load inclination factors, ground inclination factors (foundation on slope) and bearing surface inclination factors (inclined base).

Brich-Hansen (EC 7 - EC 8)

In order that a foundation may safely sustain the design load in regard to general failure for all combinations of load relative to the ultimate limit state, the following expression must be satisfied:

$$V_d \leq R_d$$

where:

- $\mathbf{V_d}$ is the bearing capacity, normal to the footing ground, including the weight of the foundation itself
- $\mathbf{R_d}$ is the foundation design bearing capacity for normal loads, also considering the eccentric and inclined loads

To better estimate $\mathbf{R_d}$ for fine grained soils short and long term situations should be considered. Bearing capacity in drained conditions is calculated by:

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

where

$A' = B' x L'$	Design effective foundation area. Where eccentric loads are involved, use the reduced area at whose center the load is applied.
c_u	Undrained cohesion.
q	Total lithostatic pressure on bearing surface
	Shape factor for rectangular foundations
	Shape factor for square or circular foundations

$i_c = 0.5\left(1+\sqrt{1-H/A' \cdot c_u}\right)$	Correction factor for the inclination due to a load H
$b_c = 1-2\alpha/(\pi+2)$	Correction factor that takes into account the inclination of the base of the foundation

Design bearing capacity 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:

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

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

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

Shape factors		Resultant inclination factors due to a horizontal load H	
$s_q = 1+\left(\frac{B'}{L'}\right)s$	rectangular	$i_q = [1-H/(V+$	$m_B = [2+(B'/L')]/[1+(B'/$ $m_L = [2+(L'/B')]/[1+(L'/$ If H forms an angle θ with $m = m_\theta = m_L \cos^2\theta + m_B \sin$
$s_q = 1+\sin\varphi'$ $s_\gamma = 0.7$	square or circular	$i_\gamma = [1-H/(V+$	
$s_\gamma = 1-0.3\left(\frac{B'}{L'}\right)$	rectangular	$i_c = i_q - (1-i_q)/$	
	rectangular, square or circular		

Correction factors proposed by Brinch-Hansen for the computation of q_{ult}

In addition to the correction factors reported in the table above will also be considered the ones complementary to the depth of the bearing surface and to the inclination of the bearing surface and ground surface (Hansen).

Meyerhof e Hanna (1978)

All the theoretical analysis about bearing-capacity calculation are based on the assumption that the soil is isotropic and homogeneous to a considerable depth but in nature, soil is generally non-homogeneous and it can be constituted by different percentages of granulometric component such as gravel, sand, silt and clay. However, the assumption of homogeneity to such soils is not strictly valid if the failure surface cuts across boundaries of distinct layers with different strength characteristics and compositions.

The present analysis is limited to a system of two distinct soil layers. For a footing located in the upper layer at a depth D, below the ground level, the failure surfaces of bearing capacity can develop on the upper layer or also involve the second layer. The condition that the upper layer is stronger than the lowest is possible and vice-versa. In both cases, a general analysis with c = 0 will be presented and will be valid also for sand or clay.

The bearing capacity of a layered system was first analyzed by Button (1953) who considered only saturated clay ($\varphi = 0$). Later on Brown and Meyerhof (1969) showed that the analysis of Button leads to unsafe results. Vesic (1975) analyzed the test results of Brown and Meyerhof and others and gave his own solution to the problem.

Vesic considered both the types of soil in each layer, that is clay and (c = 0) soils. However, confirmations of the validity of the analysis of Vesic and others are not available. Meyerhof (1974) analyzed the two layer system consisting of dense sand on soft clay and loose sand on stiff clay and supported his analysis with some model tests.
Again Meyerhof and Hanna (1978) advanced the earlier analysis of Meyerhof (1974) to encompass (c = 0) soil and supported their analysis with model tests. The present section deals briefly with the analyses of Meyerhof (1974) and Meyerhof and Hanna (1978).

Case 1: A Stronger Layer Overlying a Weaker Deposit

Figure 12.16(a) shows a footing at a depth D with a width B , in a strong soil layer (Layer 1). The depth to the boundary of the weak layer (Layer 2) below the base of the footing is H. If this depth H is insufficient to form a full failure plastic zone in Layer 1 under the bearing capacity conditions, a part of this bearing capacity will be transferred to the boundary level mn. This load will induce a failure condition in the weaker layer (Layer 2). However, if the depth H is relatively large then the failure surface will be completely located in Layer 1 as shown in Fig. 12.16b.

The ultimate bearing capacities of strip footings on the surfaces of homogeneous thick beds of Layer 1 and Layer 2 may be expressed as follows:

Layer 1

$$q_1 = c_1 N_{c1} + \frac{1}{2}\gamma_1 B N_{\gamma 1} \qquad (12.43)$$

Layer 2

$$q_2 = c_2 N_{c2} + \frac{1}{2}\gamma_2 B N_{\gamma 2} \qquad (12.44)$$

where N_{c1}, $N_{\gamma 1}$ = bearing capacity factors for soil in Layer 1 for friction angle ϕ_1

N_{c2}, $N_{\gamma 2}$ = bearing capacity factors for soil in Layer 2 for friction angle ϕ_2

For the footing founded at a depth D_f, if the complete failure surface lies within the upper stronger Layer 1 (Fig. 12.16(b)) an expression for ultimate bearing capacity of the upper layer may be written as

$$q_u = q_t = c_1 N_{c1} + q'_o N_{q1} + \frac{1}{2}\gamma_1 B N_{\gamma 1} \qquad (12.45)$$

If q1 is greater than q2 and if the depth H is insufficient to form a full failure plastic condition in Layer 1, then the failure of the footing is due to the earth pressure of soil that develops from the weakest layer toward the strongest layer. The resisting force for punching may be assumed to develop on the faces ad and be passing through the edges of the footing. The forces that act on these surfaces are (per unit length of footing),

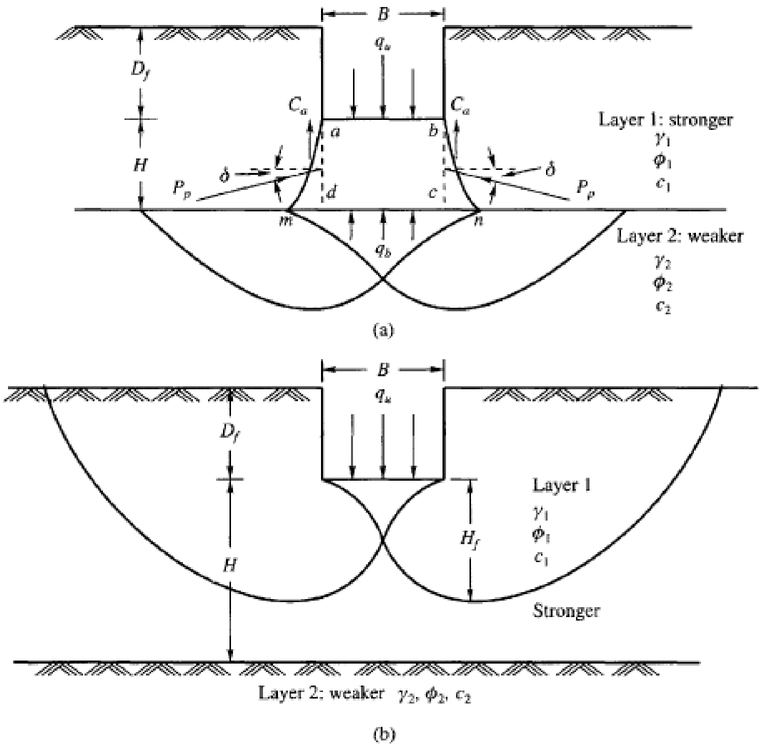


Figure 12.16 Failure of soil below strip footing under vertical load on strong layer overlying weak deposit (after Meyerhof and Hanna, 1978)

Adhesive force, $C_a = c_a H$

Frictional force, $F_f = P_p \sin \delta$ (12.46)

where c_a = unit cohesion, P_p = passive earth pressure per unit length of footing, and δ = inclination of P_p with the normal (Fig 12.16(a)).

The equation for the ultimate bearing capacity q_u for the two layer soil system may now be expressed as

$$q_u = q_b + \frac{2(C_a + P_p \sin\delta)}{B} - \gamma_1 H \tag{12.47}$$

where, q_b = ultimate bearing capacity of Layer 2

The equation for P_p may be written as

$$P_p = \frac{\gamma_1 H^2}{2\cos\delta}\left(1 + \frac{2D_f}{H}\right)K_p \tag{12.48}$$

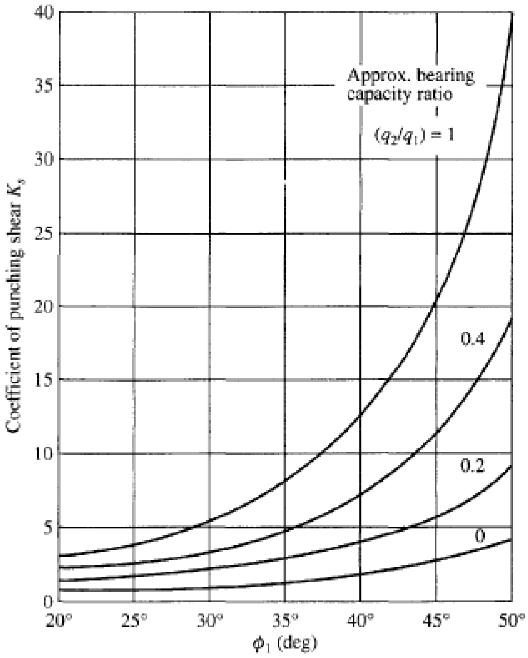


Figure 12.17 Coefficients of punching shear resistance under vertical load (after Meyerhof and Hanna, 1978)

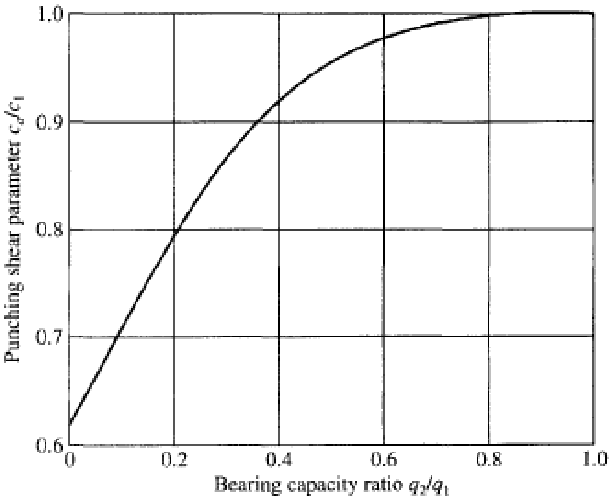


Figure 12.18 Plot of c_a/c_1 versus q_2/q_1 (after Meyerhof and Hanna, 1978)

Substituting for P_p and C_a, the equation for q_u may be written as

$$q_u = q_b + \frac{2c_a H}{B} + \frac{\gamma_1 H^2}{B} \quad 1+\frac{2D_f}{H} \quad K_p \tan\delta - \gamma_1 H \tag{12.49}$$

In practice, it is convenient to use a coefficient K_s of punching shearing resistance on the vertical plane through the footing edges so that

$$K_s \tan\phi_1 = K_p \tan\delta \tag{12.50}$$

Substituting, the equation for q_u may be written as

$$q_u = q_b + \frac{2c_a H}{B} + \frac{\gamma_1 H^2}{B} \quad 1+\frac{2D_f}{H} \quad K_s \tan\phi_1 - \gamma_1 H \le q_t \tag{12.51}$$

Figure 12.17 gives the value of K_s for various values of ϕ_1 as a function of q_2/q_1. The variation of c_a/c_i with q_2/q_1 is shown in Fig. 12.18.

Equation (12.45) for q_t and q_b in Eq. (12.51) are for strip footings. These equations with shape factors may be written as

$$q_t = c_1 N_{c1} s_{c1} + \gamma_1 D_f N_{q1} s_{q1} + \frac{1}{2}\gamma_1 B N_{\gamma 1} s_{\gamma 1} \tag{12.52}$$

$$q_b = c_2 N_{c2} s_{c2} + \gamma_1 (D_f + H) N_{q2} s_{q2} + \frac{1}{2}\gamma_2 B N_{\gamma 2} s_{\gamma 2} \tag{12.53}$$

where s_c, s_q and s_γ are the shape factors for the corresponding layers with subscripts 1 and 2 representing layers 1 and 2 respectively.

Eq. (12.51) can be extended to rectangular foundations by including the corresponding shape factors.

The equation for a rectangular footing may be written as

$$q_u = q_b + \frac{2c_a H}{B} \quad 1+\frac{B}{L} \quad + \frac{\gamma_1 H^2}{B} \quad 1+\frac{2D_f}{H} \quad 1+\frac{B}{L} \quad K_s \tan\phi_1 - \gamma_1 H \le q_t \tag{12.54}$$

Case 2: Top Layer Dense Sand and Bottom Layer Saturated Soft Clay (ϕ_2 = 0)

The value of q_b for the bottom layer from Eq. (12.53) may be expressed as

$$q_b = c_2 N_{c2} s_{c2} + \gamma_1 (D_f + H) \tag{12.55}$$

From Table (12.3), s_{c2} = (1+0.2 B/L) (Meyerhof, 1963) and N_c = 5.14 for ϕ = 0. Therefore

$$q_b = \quad 1+0.2\frac{B}{L} \quad 5.14 c_2 + \gamma_1 (D_f + H) \tag{12.56}$$

For c_1 = 0, q_t from Eq. (12.52) is

$$q_t = \gamma_1 D_f N_{q1} s_{q1} + \frac{1}{2}\gamma_1 B N_{\gamma 1} s_{\gamma 1} \tag{12.57}$$

We may now write an expression for q_u from Eq. (12.54) as

$$q_u = \quad 1+0.2\frac{B}{L} \quad 5.14 c_2 + \frac{\gamma_1 H^2}{B} \quad 1+\frac{2D_f}{H} \quad 1+\frac{B}{L} \quad K_s \tan\phi_1$$
$$+ \gamma_1 D_f \le \gamma_1 D_f N_{q1} s_{q1} + \frac{1}{2}\gamma_1 B N_{\gamma 1} s_{\gamma 1} \tag{12.58}$$

The ratio of q_2/q_1 may be expressed by

$$\frac{q_2}{q_1} = \frac{c_2 N_{c2}}{0.5\gamma_1 B N_{\gamma 1}} = \frac{5.14 c_2}{0.5\gamma_1 B N_{\gamma 1}} \tag{12.59}$$

The value of K_s may be found from Fig. (12.17).

Case 3: When Layer 1 is Dense Sand and Layer 2 is Loose Sand (c_1 = c_2 = 0)

Proceeding in the same way as explained earlier the expression for q_u for a rectangular footing may be expressed as

$$q_u = \gamma_1 (D_f + H) N_{q2} s_{q2} + \frac{1}{2}\gamma_2 B N_{\gamma 2} s_{\gamma 2}$$
$$+ \frac{\gamma_1 H^2}{B} \quad 1+\frac{B}{L} \quad 1+\frac{2D_f}{H} \quad K_s \tan\phi_1 - \gamma_1 H \le q_t \tag{12.60}$$

where $q_t = \gamma_1 D_f N_{q1} s_{c1} + \frac{1}{2}\gamma_1 B N_{\gamma 1} s_{\gamma 1}$ (12.61)

$$\frac{q_2}{q_1} = \frac{\gamma_2 N_{\gamma 2}}{\gamma_1 N_{\gamma 1}} \tag{12.62}$$

Richards et al. (1993)

Richards, Helm and Budhu (1993) developed a procedure that allows, under seismic conditions, to evaluate both the bearing capacity and induced settlements, therefore to proceed to the verification of both limit states (ultimate and damage). In this case, the calculation of the bearing capacity is performed considering the presence of inertial forces in the foundation soil due to earthquake, while the computation of settlements is obtained through an approach Newmark type. The authors have extended the classical formula of the bearing capacity as follows:

$$q_L = N_q \cdot q + N_c \cdot c + 0.5 N_\gamma \cdot \gamma \cdot B$$

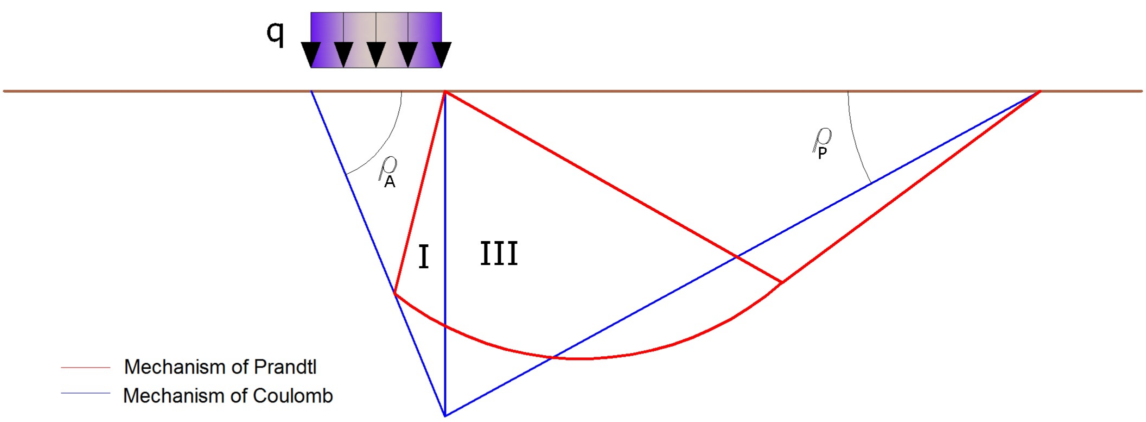
Where the bearing capacity factors are calculated using the following formulas:

$$N_c = (N_q - 1) \cdot \cot(\phi)$$

$$N_q = \frac{K_{pE}}{K_{AE}}$$

$$N_\gamma = \left(\frac{K_{pE}}{K_{AE}} - 1 \right) \cdot \tan(\rho_{AE})$$

The authors, furthermore, examined a mechanism type Coulomb with the limit equilibrium approach, considering also the acting inertia forces on the failure terrain volume . In the static field, the classical Prandtl mechanism may be approximated as shown in the figure below, removing the transition zone (range of Prandtl) considering onlythe line AC, which is considerated as an ideal wall in equilibrium under the action of active and passive thrust that it receives from I and III wedges:



Computation of bearing capacity q_{lim}

The authors have obtained the: i) the expressions of ρ_A and ρ_P angles that define the areas of active and passive thrust and ii) the active and passive thrust coefficients K_A and K_P as a function of the angle of internal friction φ of the ground and the angle of friction d ground - ideal wall:

$$\rho_A = \varphi + \tan^{-1}\cdot\left\{\frac{\sqrt{\tan(\varphi)\cdot(\tan(\varphi)\cdot\cot(\varphi))\cdot(1+\tan(\delta)\cdot\cot(\varphi))} - \tan(\varphi)}{1+\tan(\delta)\cdot(\tan(\varphi)+\cot(\varphi))}\right\}$$

$$\rho_P = \varphi + \tan^{-1}\cdot\left\{\frac{\sqrt{\tan(\varphi)\cdot(\tan(\varphi)\cdot\cot(\varphi))\cdot(1+\tan(\delta)\cdot\cot(\varphi))} + \tan(\varphi)}{1+\tan(\delta)\cdot(\tan(\varphi)+\cot(\varphi))}\right\}$$

$$K_A = \frac{\cos^2(\varphi)}{\cos(\delta)\left\{1+\sqrt{\frac{\sin(\varphi+\delta)\cdot\sin(\varphi)}{\cos(\delta)}}\right\}^2}$$

$$K_P = \frac{\cos^2(\varphi)}{\cos(\delta)\left\{1-\sqrt{\frac{\sin(\varphi+\delta)\cdot\sin(\varphi)}{\cos(\delta)}}\right\}^2}$$

It is possible to observe that the use of previous equations assuming $\varphi = 0.5\delta$, leads to the value of the bearing capacity factors very close to those based on an analysis of Prandtl. Richards et. al (1993) used the Coulomb mechanism in seismic case, taking into account the inertia forces acting on the ground volume in failure. These mass forces, due to acceleration kh g and kv g, respectively acting in horizontal and vertical direction, are equal to kh g and kv g. Were thus obtained the extensions of the expressions of ρ_A and ρ_P, as well as K_A and K_P, respectively indicated as ρ_{AE} and ρ_{PE}, and K_{AE} and K_{PE} to denote the seismic conditions:

$$\rho_{AE} = (\varphi-\vartheta) + \tan^{-1}\cdot\left\{\frac{\sqrt{(1+\tan^2(\varphi-\vartheta))\,[1+\tan(\delta+\vartheta)\cdot\cot(\varphi-\vartheta)]} - \tan(\varphi-\vartheta)}{1+\tan(\delta+\vartheta)\cdot(\tan(\varphi-\vartheta)+\cot(\varphi-\vartheta))}\right\}$$

$$\rho_{PE} = (\varphi-\vartheta) + \tan^{-1}\cdot\left\{\frac{\sqrt{(1+\tan^2(\varphi-\vartheta))\cdot[1+\tan(\delta+\vartheta)\cdot\cot(\varphi-\vartheta)]} - \tan(\varphi-\vartheta)}{1+\tan(\delta+\vartheta)\cdot(\tan(\varphi-\vartheta)+\cot(\varphi-\vartheta))}\right\}$$

$$K_{AE} = \frac{\cos^2(\varphi-\vartheta)}{\cos(\vartheta)\cdot\cos(\delta+\vartheta)\left\{1+\sqrt{\frac{\sin(\varphi+\delta)\cdot\sin(\varphi-\vartheta)}{\cos(\delta+\vartheta)}}\right\}^2}$$

The values of Nq and Nγ can also be determined using the above formulas, of course, using the expressions of the ρ_A and ρ_P angles and of K_{AE} and K_{PE} coefficients related to the seismic case. In these expressions appears the angle θ defined as:

$$\tan(\theta) = \frac{k_h}{1 - k_v}$$

The table below shows the bearing capacity factors calculated for the following parameter values:

$$\varphi = 30°$$
$$\delta = 15°$$

And for several values of the seismic thrust coefficients:

kh/(1-kv)	Nq	Ng	Nc
0	16.51037	23.75643	26.86476
0.087	13.11944	15.88906	20.9915
0.176	9.851541	9.465466	15.33132
0.268	7.297657	5.357472	10.90786
0.364	5.122904	2.604404	7.141079
0.466	3.216145	0.879102	3.838476
0.577	1.066982	1.103E-03	0.1160159

Table of bearing capacity factors for φ=30°

Bearing capacity for foundations on rock

Where foundations rest on rock, it is appropriate to take into consideration certain other significant parameters such as the geologic characteristics, type of rock and its quality measured as RQD. It is the practice to use very high values of safety factor for bearing capacity of rock and correlated in some way with the value of **RQD** (Rock quality designator). For example for a rock whose RQD is up to a maximum of 0.75 the safety factor oscillates between 6 and 10. Terzaghi's formula can be used in calculation of rock bearing capacity using friction angle and cohesion of the rock or those proposed by Stagg and Zienkiewicz (1968) according to which the coefficients of the bearing capacity are:

$$N_q = \tan^6\left(45 + \frac{\varphi}{2}\right)$$

$$N_\gamma = N_q + 1$$

These coefficients should be used with form factors from the formula of Terzaghi. Ultimate bearing capacity is a function of RQD as follows:

$$q' = q_{ult} \cdot (RQD)^2$$

If rock coring does not render whole pieces (RQD tends to 0) the rock is treated as a soil estimating as best the factors: c and φ.

Sliding check

The stability of a foundation should be verified with reference to collapse due to sliding as well as to general failure. For collapse due to sliding, the resistance is calculated as the sum of the adhesion component and the soil-foundation friction component. Lateral resistance arising from passive thrust of the soil can be taken into account using a percentage supplied by the user.Resistance due to friction and adhesion is calculated with the expression:

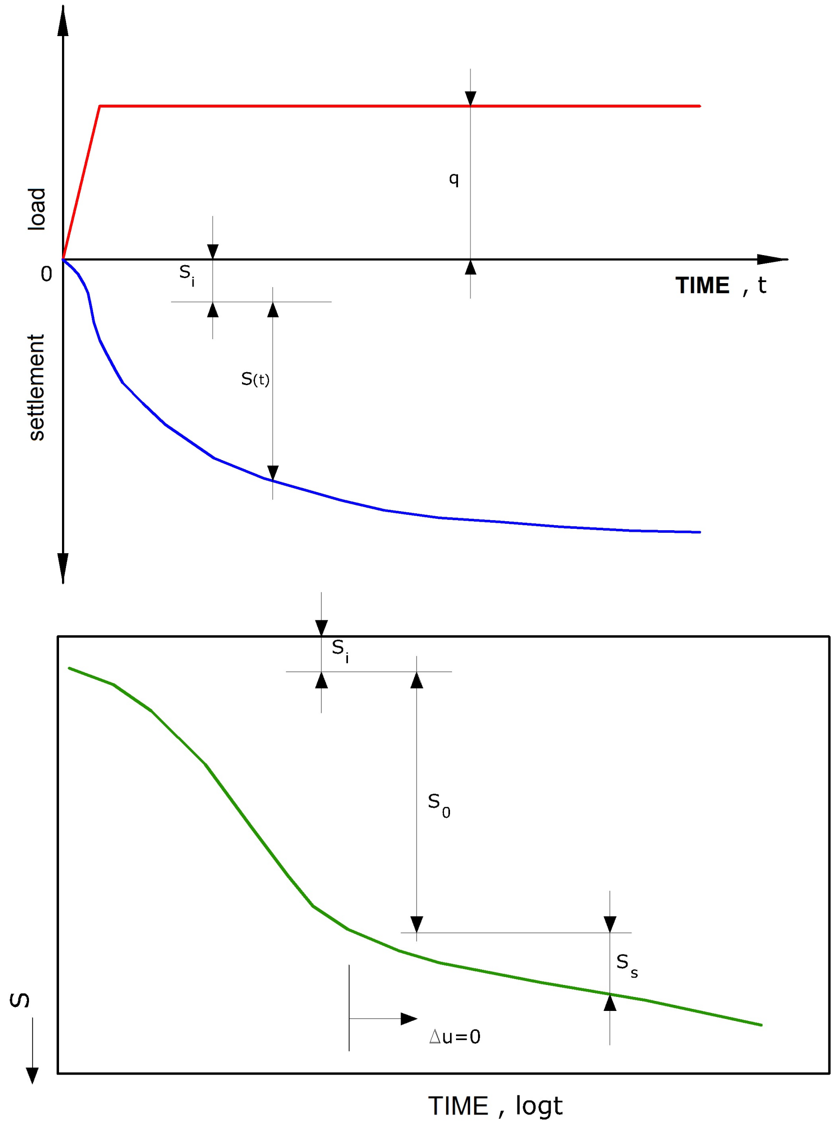
$$F_{Rd} = N_{sd} \cdot \tan\delta + c_a \cdot A'$$

where

N_{sd}	is the value of the vertical force
δ	is the angle of shearing resistance at the base of the foundation
c_a	is the foundation-soil adhesion
A'	is the effective foundation area; where eccentric loads are involved, use the reduced area at whose center the load is applied

Computation of settlements

The application of a load of finite size on a cohesive soil generates a series of phenomena which can be summarized as illustrated in image below:



Settlement types

1. During the loading phase in the ground develop of the overpressures of interstitial water Δu, and due to the low permeability of the soil is permissible to assume that, in the context of the usual speed of application of the load, there are undrained conditions. The clay layer deforms in volume almost constant and the settlement that follows is indicated as *immediate settlement*.
2. The establishment of drainage, with the progressive transfer of load from the fluid phase to the solid skeleton, implies further settlements, the speed of which in time is primarily related to the drainage conditions. The process is known as primary consolidation, the analysis is conducted with the various models of the theory of consolidation. The settlement that follows in this process of expulsion of water from the interstitial voids is indicated as *consolidation settlement*.
3. Finally, even when the interstitial surcharges are dissipated (Δu = 0), there continue to be settling in time due to creep in drained conditions, and the settlement is known as *secondary settlement*.

Elastic settlements

The settlement of a rectangular foundation of size B' × L' placed on the surface of an elastic support may be calculated by use of an equation based on the elasticity theory (Timoshenko and Goodier (1951)):

$$\Delta H = q_0 \cdot B' \frac{1-\mu^2}{E_s}\left(I_1 + \frac{1-2\mu}{1-\mu} \cdot I_2\right) \cdot I_F \qquad (1)$$

where:

q_0 Intensity of contact pressure.

B' Minimum size of reactant area.

E_s and ν Elasticity parameters for soil.

I_i =f(L'/B', H, ν, D) Influence factors that depend on: L'/B', thickness of layer H, Poisson's ratio m, bearing surface depth D

I_F influence factor

Coefficients I_1 & I_2 may be calculated using the equations of Steinbrenner (1934) (Bowles), as functions of the relation M=L'/B' and N=H/B, using B'=B/2 e L'=L/2 for coefficients relative to the center and B'=B & L'=L for coefficients Ii at the edge.

Influence factor IF is due to *Fox (1948)*, and suggests that settlement is reduced with depth, depending on Poisson’s ratio m, and of the ratio L/B.

In order to simplify the equation (1) the coefficient I_S is introduced:

$$I_S = I_1 + \frac{1-2\mu}{1-\mu} \cdot I_2$$

Relationship (1) can be written as:

$$\Delta H = q_0 \cdot B' \frac{1-\mu^2}{E_S} I_S \cdot I_F$$

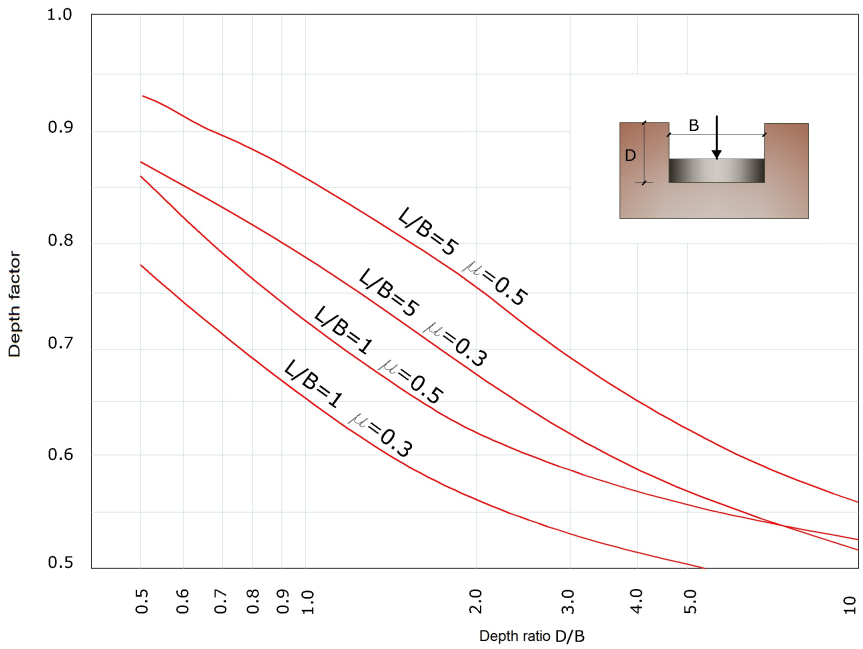
The equation can be applied to flexible or rigid foundations with appropriate changes in the value of I_s.

The author, analyzing a number of cases, has concluded that the equation formulated previously, in order to to provide good results must, be applied as follows:

1. Make the best estimate of q_0
2. Convert the foundation, if circular, in an equivalent square foundation
3. Determine the point where to calculate the settlement and divide the support base so that the point is in correspondence with one of the outer edge or an inner edge common to more rectangles
4. The thickness H of the layer responsible for the settlement should be taken as the minimum of the two

following values: depth z = 5B where B is the minimum overall size of the base of the foundation; depth at which is located a hard layer (E_S of the layer must be about 10 times the value of the adjacent thickness).

5. Correctly calculate the ratio H/B'. For a layer thickness H=z=5B we find, for the center of the foundation, H/B'=5B/0,5B=10B, for a point 5B/B=5
6. Obtain Is having an accurate calculation of m and obtaining the influence coefficients I_1 and I_2 from the table proposed by the same author
7. Obtain I_F with the help of the image below
8. Obtain E_S in the thickness of the layer z=H as weighted average of the values of E_{SI} of single layers in the thickness H_I

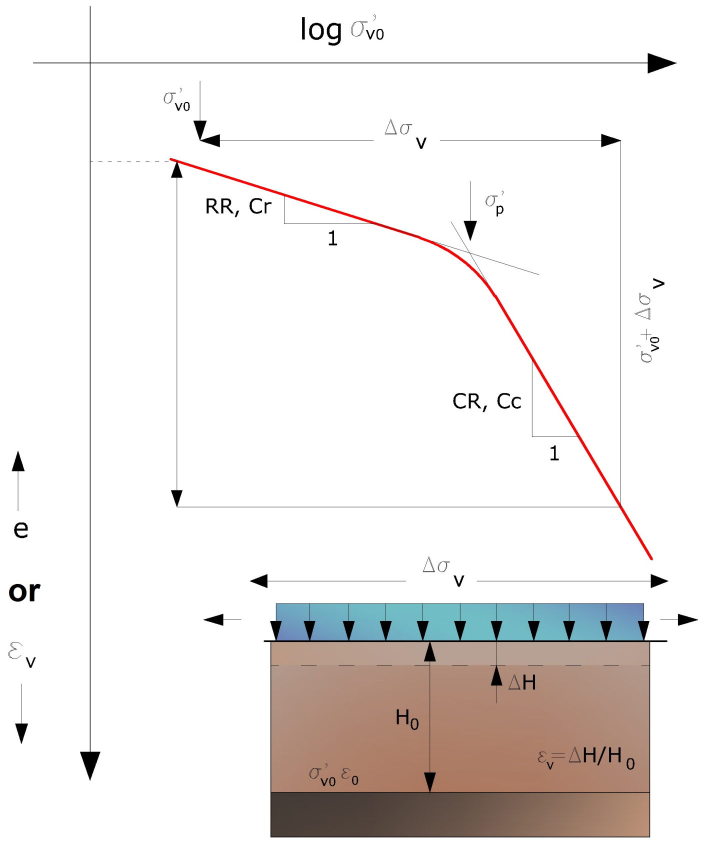


Influence factor I_F for a foundation calculated at a depth D

Oedometric settlements

The computation of settlement with the oerdometric approach allows the evaluation of monodimensional settlement *(Terzaghi-1943)*, produced by stresses induced by the application of a load in conditions of inhibited lateral expansion. However the computation with this method is to be considered empiric rather than theoretic. Nonetheless the ease of use and of controlling the influence of the various parameters involved make of this a very widespread method.With reference to the image below the settlement ΔH of a layer of an initial thickness H_0 is given by:

$$\Delta H = H_0 \cdot \left[RR \cdot \log \frac{\sigma'_p}{\sigma'_{v0}} + CR \cdot \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p} \right]$$



Oedometric settlement

The process of settlement calculation goes through two phases:

1. Calculation of vertical stresses induced at various depths applying the theory of elasticity *(Boussinesq, Westergaard, etc.)*
2. Evaluation of compression parameters through an oedometric test

With reference to the results of the oedometer test, the settlement is evaluated as:

$$\Delta H = H_0 \cdot RR \cdot \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

If the soil is overconsolidated (OCR>1), that is if the increment of stresses due to the application of load does not cause preconsolidation pressure to exceed σ'p ($\sigma'_{v0} + \Delta\sigma_v < \sigma'_p$)
If on the other hand the soil is normally consolidated ($\sigma'_{v0}=\sigma'_p$), deformation occurs in the compression interval and the settlement is calculated as:

$$\Delta H = H_0 \cdot CR \cdot \log \frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

where:

RR	Recompression ratio
CR	Compression ratio
H_0	Initial layer thickness
σ'_{v0}	Effective vertical stress before the application of the load
$\Delta\sigma_v$	Increment in vertical stress due to the application of the load

As an alternative to parameters RR and CR one can refer to the oedometric modulus M, however, in such case, it will be necessary to use judgment in selecting the value to use taking account the stress interval (σ'_{v0}+ $\Delta\sigma_v$) significant for the problem in question.

The correct application of this approach requires:

- Subdivision of compressible layers into smaller ones (max. 2.00m)
- An estimate of the oedometric modulus for each layer
- Computation of settlement as a sum of the contribution of each subdivision of small layers

Many use the formulas above to calculate settlement both for clays and sands with fine to medium granularity as the elasticity modulus is derived directly from consolidation tests. However for soils with a coarser grain, the dimensions of oedometric testers are not very significant for the global behaviour of the layer and for sands it is advisable to use penetration tests either static or dynamic.

Secondary settlement

Secondary settlement is calculated by the expression:

$$\Delta H_S = H_c \cdot C_\alpha \cdot \log \frac{T}{T_{100}}$$

where:

H_c	is the height of the layer in phase of consolidation;
C_a	is the coefficient of secondary consolidation as vector of the secondary portion of the curve Settlement-logarith time;
T	time for which the settlement is required;
T_{100}	time for the completion of primary settlement.

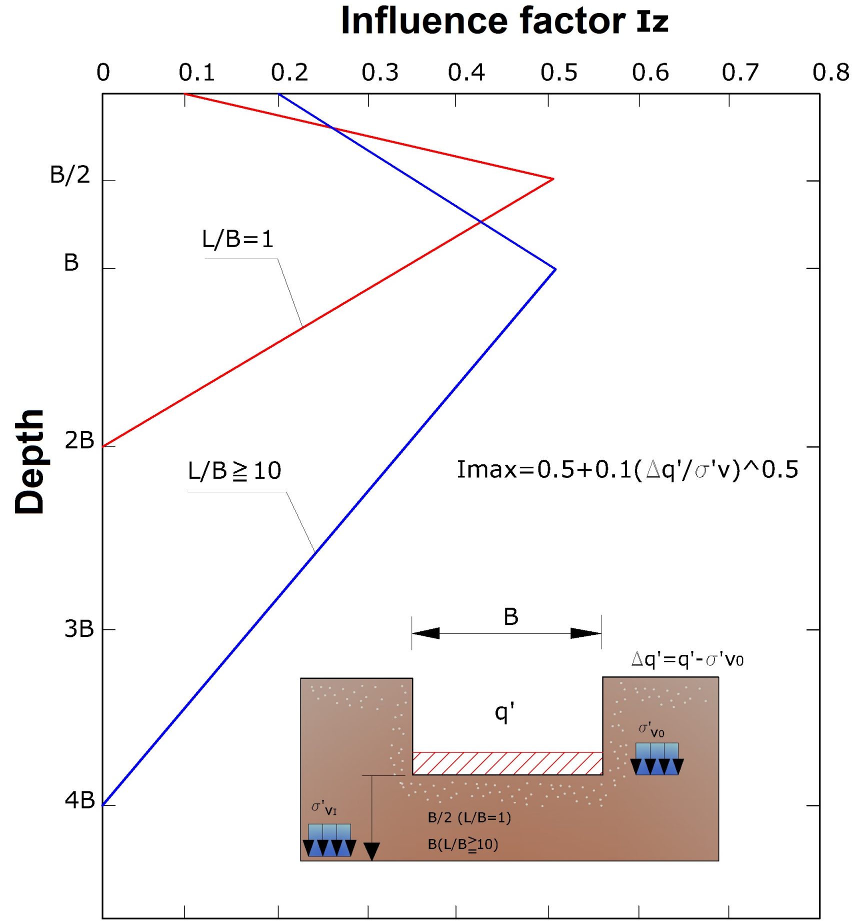
The assumptions at the base of this method are:

- the secondary consolidation starts after the exhaustion of the primary consolidation process;

- the value of C_a can be considered constant during the evolution of the secondary settlement.

Schmertmann settlements

Schmertmann (1970) proposed an alternative method of calculating settlement as related to the variation to the pressure bulb on deformation. Thus Schmertmann proposes a triangular deformation diagram where the depth at which significant deformation occurs is 4B for strip foundations and 2B for square or circular foundations.



Variation of the influence factor with depth

With this approach settlement is expressed by the following expression:

Where:

Δq is the net load applied to the foundation;

Iz is a deformation factor whose value is null at depth 4B or 2B respectively for Strip or Round/Square foundations.

$$I_{z\,max} = 0.5 + 0.1\left(\frac{\Delta q}{\sigma'_{vi}}\right)^{0.5}$$

Where:

σ'_{v0} is the effective vertical stress at depths of B/2 for round or square foundations and at depth B for strip foundations

E_i is the modulus of soil deformation corresponding to the i-th layer considered in the calculation;

Δz is the thickness of the i-th layer;

C_1 and C_2 are two correction factors.

Modulus E_i is assumed as 2.5 qc for square/round foundations and 3.5 qc for strip foundations. For intermediate cases the value is interpolated dependent on the value of L/B.
The term qc in the determination of E_i is the CPT tip resistance.The expressions for C_1 and C_2 are:

$C_1 = 1 - 0.5\dfrac{\sigma'_{v0}}{\Delta q} > 0.5$	That accounts for footing depth
$C_2 = 1 + 0.2 \cdot \log\dfrac{t}{0.1}$	That accounts for the deformations, different in time, due to secondary effect

Where t represents the time in years, after completion of the structure, for which settlement is calculated.

Burland and Burbidge settlements

There where dynamic penetration test results are available, it is possible to rely on Burland and Burbidge (1985) method for settlement computation for which an index of compressibility Ic is correlated to the result NSPT of the dynamic penetration test. The formula proposed by the authors is:

Where:

q'	gross effective pressure;
σ'_{v0}	effective vertical stress at footing depth;
B	width of the foundation;
I_c	compressibility index;

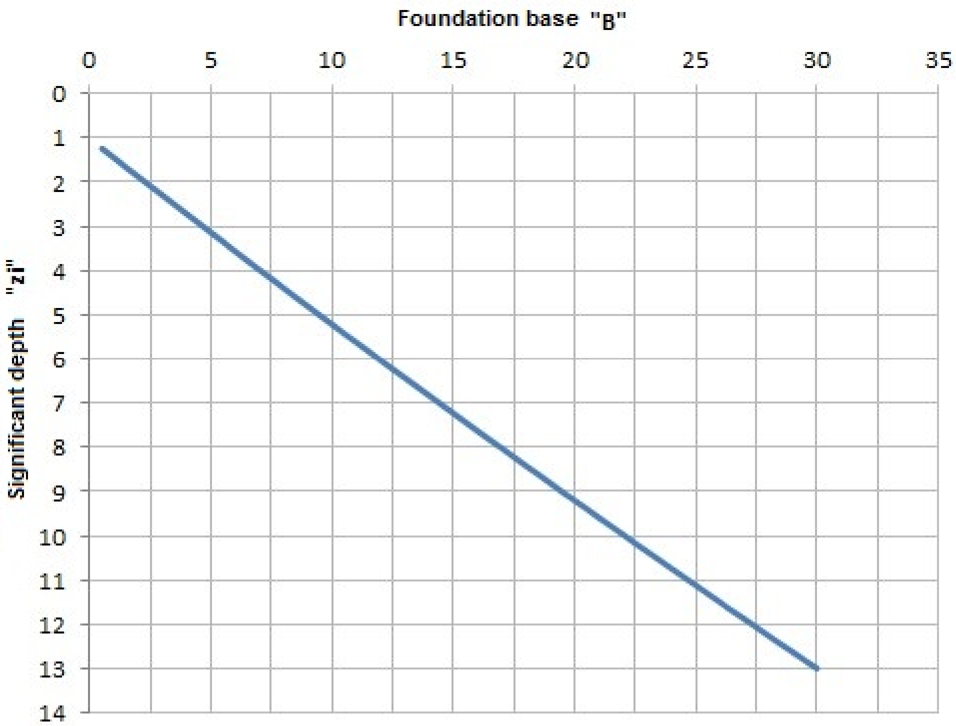
f_s, f_H, f_t corrective factors that account respectively for the form, compressible layer thickness and time, for the viscous component

Ic, compressibility index is related to the average value N_{AV} of N_{SPT} at a significant depth z_i:

$$I_C = \frac{1.076}{N_{AV}^{1.4}}$$

To calculate the value of z_i it is used the following relationship:

$$z_i = 1.025 + 0.4286 \cdot B - 0.0001 \cdot 9.91 \cdot B^2$$



Trend of the significant depth as a function of the foundation base

As regards the N_{SPT} values to use in calculating the average N_{AV}, it is opportune to remember that values should be corrected for sands with silt content under the water table and N_{SPT} >15 as indicated by Terzaghi & Peck (1948):

Where Nc is the corrected value to use in calculation.

For gravel or gravelly sandy deposits, the corrected value is:

For corrective factors f_S, f_H ed f_t the expressions are:

$$f_S = \left(\frac{1.25 \cdot L/B}{L/B + 0.25} \right)^2$$

$$f_H = \frac{H}{z_i} \left(2 - \frac{H}{z_i} \right)$$

$$f_t = \left(1 + R_3 + R \cdot \log \frac{t}{3} \right)$$

Where
t = time in years > 3;
R_3 = a constant of value 0.3 for static loads and 0.7 for dynamic loads;
R = a constant of value 0.2 for static loads and 0.8 for dynamic loads.

1.9.1 Seismic correction factors PAOLUCCI & PECKER

The effect of the seismic action on the bearing capacity of a foundation can be evaluated by introducing for the three terms of the bearing capacity the partial correction factors z calculated as follows:

$$z_q = \left(1 - \frac{k_h}{tg\phi} \right)^{0,35}$$

$$z_c = 1 - 0,32 \cdot k_h$$

$$z_\gamma = z_q$$

where:

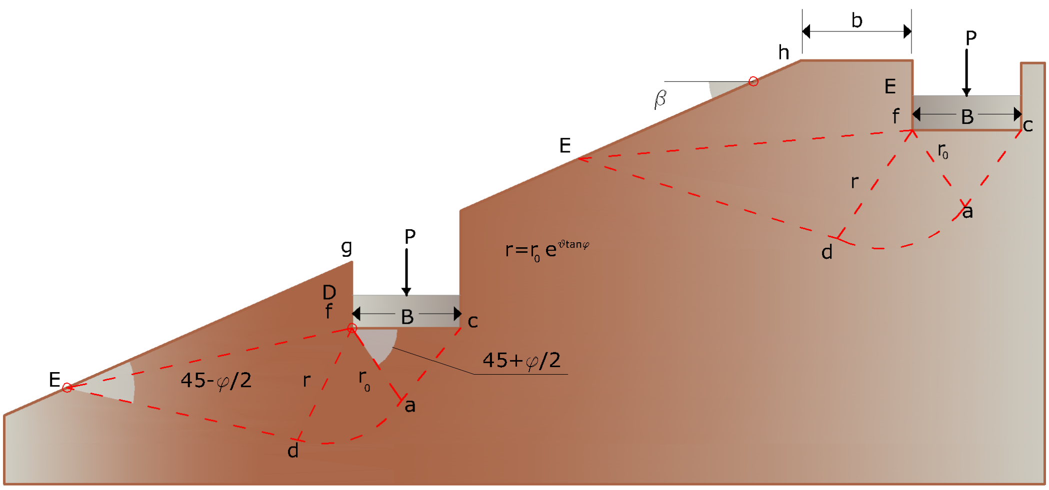
is the horizontal seismic coefficient calculated according to the maximum ground acceleration a_g/g and the soil category.

See also:

[Computation of bearing capacity factors Nq, Nc, Nγ in seismic conditions]

1.9.2 Bearing capacity of footings on slopes

A special problem that may be encountered occasionally is that of a footing located on or adjacent to a slope (image below). From the figure it can be seen that the lack of soil on the slope side of the footing will tend to reduce the stability of the footing.



Foundation set up on a slope or in the immediate vicinity

The solution of this problem is resolved by calculating reduced coefficients N'c and N'q and assuming that the slope line is a principal direction.

The computation of N'c reduced considers as failure surface ade = L_0 related to the case of foundation in horizontal plane, and and the failure surface adE = L_1 of the image above obtaining:

$$N_c^{'} = N_c \cdot \frac{L_1}{L_0}$$

The coefficient N'q is reduced using the ratio of the areas D(ce) = A_0, for a level footing, and Efg of the image above (or the alternative Efgh = A_1 of the image above) to obtain the following:

$$N_q^{'} = N_q \cdot \frac{A_1}{A_0}$$

In case of slope with $A_1 \geq A_0$:

$$N_q^{'} = N_q$$

The overall slope stability should be checked for the effect of the footing load using a slope-stability program (Slope by GeoStru).

The bearing capacity is computed using the usual equations and reduction coefficients:

$$q_{lim} = \gamma \cdot D \cdot N_q^{'} \cdot s_q \cdot i_q + c \cdot N_c^{'} \cdot s_c \cdot i_c + \frac{1}{2}\gamma \cdot B \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma$$

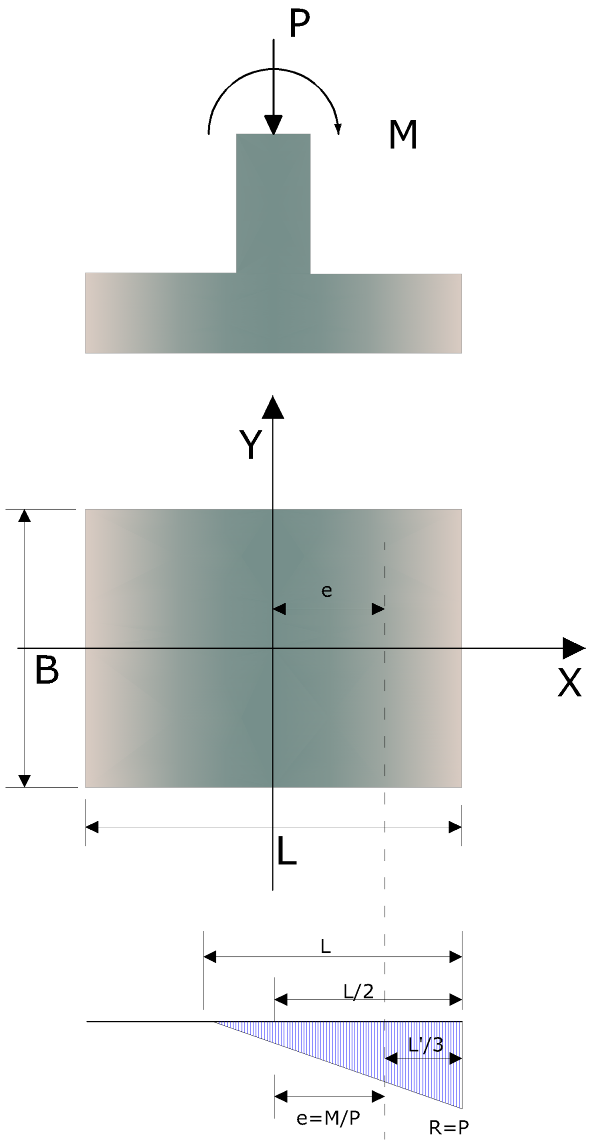
The coefficient Nγ ,that is a function of the weight of the soil, is not corrected to account for the slope. When β = 0 the coefficients N'c and N'q are the same as those acting when the footing is on leveled ground for every value of f independently of the ratios D/B (bearing surface depth/width) and b/B (distance from slope/width). When D/B>0 the effect of the depth is already included in N_c and N_q so the reduction coefficients should no longer be used.

1.9.3 Computation of soil pressures

There are situations where it is impossible to maintain the resultant of the soil pressure within the middle third of the foundation base. This situation occurs when one or more combinations of load exceeds substantially the ability of the foundation to resist overturning moment *(temporary or transient load conditions due to wind or earthquake)*. Although the foundations are usually not designed for such loading conditions, their stability to overturning should be tested in the presence of these temporary loads.
The geotechnical consultant should provide, on request, a separate assessment of the allowable pressure on the soil q'_0 valid for temporary load conditions, in addition to the one used for the operating conditions.

For an eccentricity

regarding one axis, we obtain an equation from the following figure to determine the maximum pressure on the soil and the effective length L' of the foundation, where it is obvious that the base area is not reacting for a length equal to L - L'.



Pressure diagram when e>L/6

The area of the triangle of pressures must be equal to the vertical load P and the resultant must be applied at L'/3 of the most stressed edge and pass through the barycenter of the triangle. This point is located at a distance

$$e = \frac{M}{P}$$

from the center of the foundation so

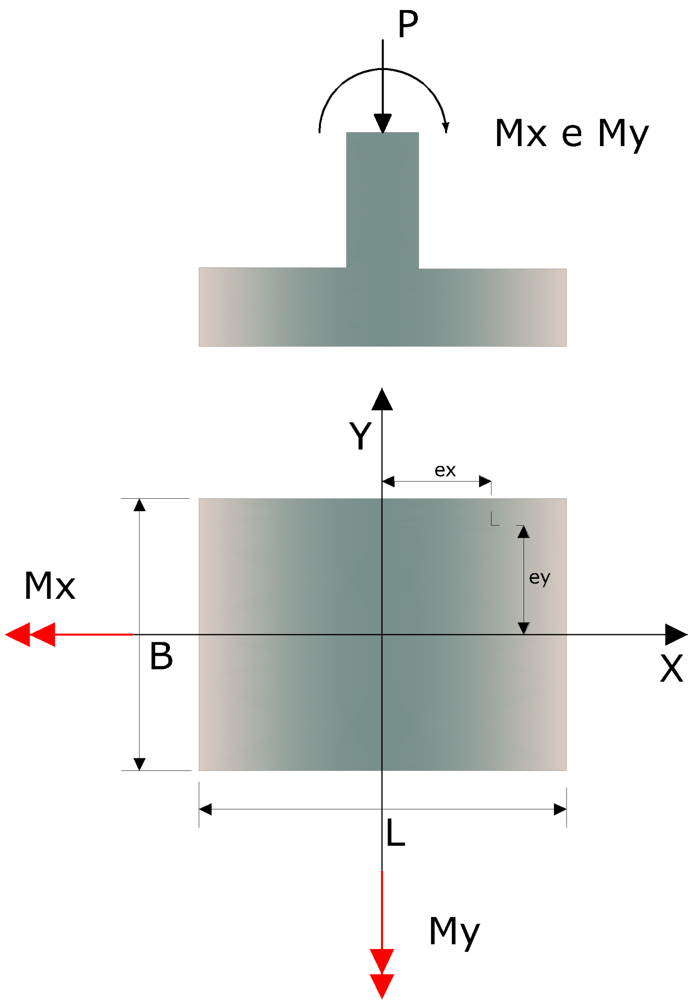
and

Substituting L' in the expression of P and solving for q we obtain

$$q = \frac{2P}{3B(L/2 - e)} \leq q'_a$$

With P, q'_a and eccentricity e fixed, is solved relating to B and L for attempts until to satisfy the equality.

When the moment is present, related to both x and y axis, the position of the resultant is like the in figure below



Position of the resultant when we have moment with respect to x and with respect to y

and if both eccentricities are

$$e_x > \frac{L}{6} \quad e_y > \frac{B}{6}$$

only a part of the foundation is reacting.
The pressure on the soil for foundations with eccentricity regarding both axes can be calculated, when there is no lifting of the foundation from the soil, as follows:

$$q = \frac{P}{A} \pm \frac{M_y x}{I_y} \pm \frac{M_x y}{I_x}$$

or

$$q = \frac{P}{BL}\left(1 \pm \frac{6e_x}{L} \pm \frac{6e_y}{B}\right)$$

We remind that

$$I_x = \frac{LB^3}{12} \quad I_y = \frac{BL^3}{12}$$

M_y is the moment regarding the y axis;
M_x is the moment regarding the x axis;

$$e_x = \frac{M_y}{P} \quad e_y = \frac{M_x}{P}$$

The positive sides are the ones shown.

In the case of circular foundations the relationships used for the calculation of the maximum contact pressure are:

$$q = \frac{P}{A} \pm \frac{M_x \cdot y}{I_x} \pm \frac{M_y \cdot x}{I_y}$$

$$A = \pi \cdot R^2$$

$$I_x = I_y = \frac{\pi \cdot R^4}{4}$$

y and x are calculated from the axis center of gravity of the section. Imposing q = 0 is found the position of the neutral axis that allows to calculate the effective area (area reacting to compression) used in the sliding check.

1.9.4 Check limit load (SLU)

The vertical bearing capacity of the foundation soil is verified according to the theory of limit states using the following inequality:

$$\sigma \le \frac{R_d}{\gamma_{RV}}$$

or based on the factor of safety as:

$$\frac{R_d}{\sigma} \ge SF_v$$

where:

σ-extreme design contact stress at the footing bottom

R_d-design bearing capacity of foundation soil

γ_{RV}-coefficient of vertical bearing capacity of foundation

SF_v-safety factor for vertical bearing capacity

Extreme design contact stress at the footing bottom is assumed the form:

$$\sigma = \frac{V}{A_{ef}}$$

where:

V-extreme design vertical force

A_{ef}-effective area of foundation

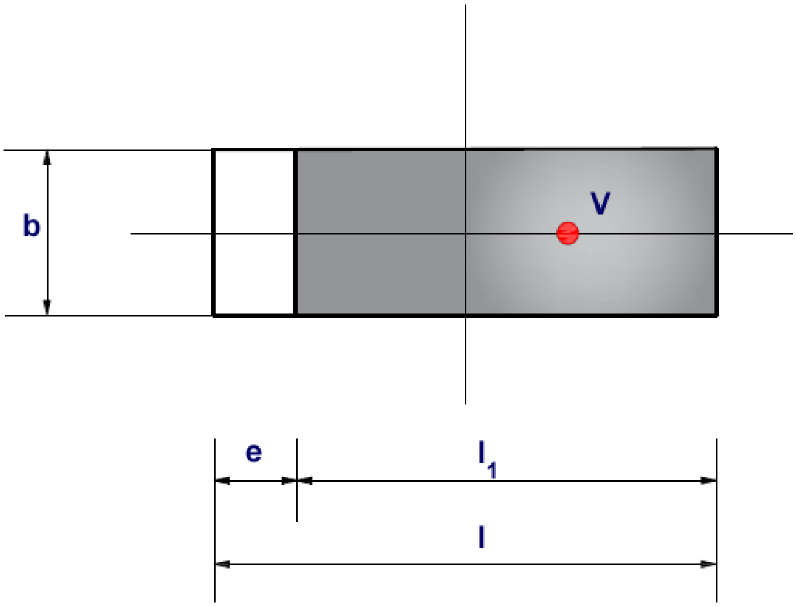
Effective Area

When solving the problem of eccentrically loaded foundations the program offers two options to deal with an effective dimension of the foundation area:

- a rectangular shape of effective area is assumed
- circular shape of effective area is assumed

Rectangular shape

A simplified solution is used in such cases. In case of axial eccentricity (bending moment acts in one plane only) the analysis assumes a uniform distribution of contact stress σ applied only over a portion of the foundation l_1, which is less by twice the eccentricity e compared to the total length l.



Determination of effective area in case of axial eccentricity

An effective area ($b*l_1$) is assumed to compute the contact stress, so that we have:

$$\sigma = \frac{V}{b.(l-2.e)}$$

In case of a general eccentric load (foundation is loaded by the vertical force V and by bending moments M_1 and M_2 the load is replaced by a single force with given eccentricities:

$$e_1 = \frac{M_1}{V}$$

$$e_2 = \frac{M_2}{V}$$

The size of effective area follows from the condition that the force V must act eccentrically:

$$A_{ef} = b_{ef} . l_{ef} = (b - 2.e_2).(l - 2.e_1)$$

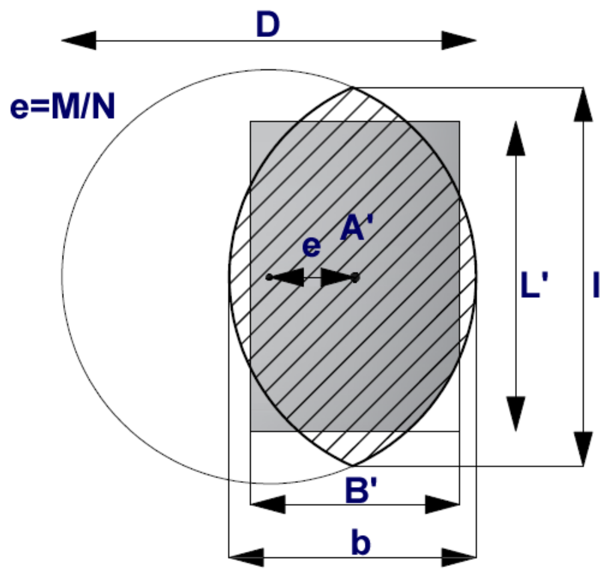
Circular shape

A circular foundation subjected to a vertical load applied with an eccentricity e=Md/Nd can be regarded as an equivalent fictitious foundation with a centrally applied load (Figure), as suggested by Meyerhof (1953) and Vesic (1973). In this case,the area of the fictitious foundation, A', can be calculated as:

$$A' = \frac{D^2}{2}\left(\arccos\frac{2e}{D} - \frac{2e}{D}\sqrt{1-\left(\frac{2e}{D}\right)^2}\right)$$

The aspect ratio of the equivalent rectangular area can also be approximated as the ratio of the line lengths b to l, as shown in Figure that is,

$$\frac{B'}{L'} = \frac{b}{l} = \sqrt{\frac{D-2e}{D+2e}}$$



Calculating method of the equivalent dimensions of a circular foundation subjected to a non-barycentric load.

1.9.5 Post-Seismic settlements

The cause of reconsolidation settlements registered after a seismic event in a soil is due to dissipation of the pore water pressure as the water is expelled from the concerned area.To estimate the magnitude of this settlement is necessary to characterize the various soil layers from a geotechnical point of view through in situ and laboratory tests.The number of investigated verticals will be more larger the more important is the work to be carried out and how extensive is the area of investigation. It is necessary to prepare with appropriate surveys the extent of fluctuations in groundwater levels and consider in the analysis the less precaution condition.
For each of the investigated verticals will be assessed the post cyclic reconsolidation settlements. The reconsolidation settlement, for **granular saturated liquefiable soils** and for *cohesive soils*, can be calculated using the following expression:

$$\Delta H = \varepsilon_{vr} \cdot H$$

where H is the height of the generic layer and ε_{vr} (%) represents the post-cyclic volumetric strain defined by:

$$\varepsilon_{vr} = \frac{\alpha \cdot C_r}{1 + e_0} \log\left(\frac{1}{1 - \frac{\Delta u}{\sigma'_0}}\right)$$

where:

α	experimental constant between 1 and 1.5
e_0	the initial void ratio
$C_r = 0.225\ C_c$	the post-cyclic reconsolidation ratio
C_c	compression ratio

Note: There are some empirical relations that allow to evaluate, in an approximate way, the compression ratio. In the case of granular soils are functions of the relative density, in the case of cohesive soils are functions of the plasticity index.

In the case of cohesive soils, Loadcap calculates the ratio of pore water pressure as:

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

where

σ'_v	is the initial value of the effective mean pressure at the considered depth

σ'_{v0} is the effective vertical pressure and k_0 the thrust coefficient at rest

γ_{max} is the maximum shearing strain reached during the earthquake
β is taken equal to 0.45 (experimental coefficient)
γ_v is the volumetric deformation threshold, determined by cyclic laboratory tests

Ma can also be evaluated, in first approximation, with the relationship that follows:

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

OCR is the overconsolidation ratio, A and B are the experimental coefficients that can be calculated by linear interpolation from the table below:

I_P [%]	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 the coefficients A and B

The ratio of pore water pressure, in the case of loose liquefiable soils, is determined by linear interpolation from the values reported in table below, depending on the amplitude of the maximum deformation induced by the ground.

γ_{max} [%]	$r_u = \Delta_u / \sigma'_0$
0.005	0.2
0.1	0.4
0.2	0.6
0.4	0.8
5	0.95

Pore water pressure ratio r_u as a function of γ_{max}

The amplitude of the maximum shearing strain γ_{max} is calculated from the following relationship:

where

$a_{max,s}$ peak acceleration at the ground level of the design earthquake
g acceleration of gravity

σ_v total vertical pressure

r_d reduction factor of the seismic action that puts into account the deformation of the subsoil determined by the relation rd=1-0.015z;

G shear modulus corresponding to the strain level gmax

The shear modulus can be determined from laboratory tests or using the table below by applying a reduction factor to the shear modulus G_0 (shear modulus at low strains).

$a_{max,s}$ *[g]*	G/G_0
0.10	0.80
0.20	0.50
0.30	0.35
0.40	0.28

Reduction factor of the shear modulus in the first 20 m as a function of the acceleration $a_{max,s}$

Settlement computation induced by the earthquake in saturated granular soils

The volumetric strain ev in saturated granular soils can be estimated from CPT tests as a function of normalized and corrected tip resistance, (qc1N) cs and of the safety factor to liquefaction FL, and from SPT tests as a function of the normalized and corrected SPT resistance (N1)60,cs and the cyclic stress ratio CSR.
The post-seismic settlement for each layer is given by:

$\Delta si = \varepsilon vi * \Delta zi.$

Alternatively, the volumetric strain, ε_v (expressed in decimals), can be estimated by the following expressions (Idriss and Boulanger, 2008):

$$\varepsilon_v = 1.5 \cdot exp\left(2.551 - 1.147 \cdot (q_{c1Ncs})^{0.264}\right) \cdot min\,(0.08, \gamma_{max}) \quad \text{con } qc1Ncs \geq 21$$

$$\varepsilon_v = 1.5 \cdot exp\left(-0.369 \cdot \sqrt{(N_1)_{60cs}}\right) \cdot min\,(0.08, \gamma_{max})$$

where γmax (decimal) is the maximum shear strain induced by the seismic action, determinable, in a first approximation, with the empirical relation:

$$\gamma_{max} = \frac{a_{maxs}}{g} \cdot \sigma_{v0} \cdot r_d \cdot \frac{1}{G}$$

where G is the shear modulus of deformation corresponding to the level γ, which can be determined by an iteration process, knowing the value of the initial stiffness G0 (= ρVs^2) using the law of variation G(γ)/G0 obtained with dynamic tests in the laboratory or, in an approximate way, deduced from literature curves for soils with similar properties (see eg. Figure 4 from AGI, 2005).

Settlement computation induced by the earthquake in unsaturated granular soils

The settlement induced by the seismic action in unsaturated granular soils can be estimated from the results of SPT tests with the method Pradel (1998), as follows:

$\Delta si = 2\varepsilon_{NC} * \Delta zi$ where Δs is the settlement of the layer of a thickness Δz

The latter can be determined, in first approximation, with the empirical relation:

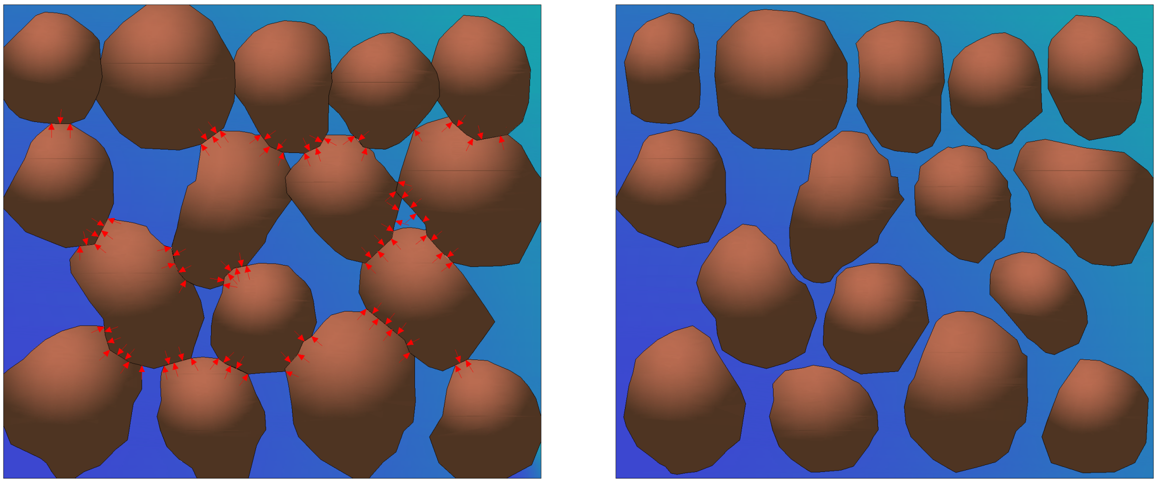
$$\gamma = 0.65 \cdot \frac{a_{maxs}}{g} \cdot \sigma_{v0} \cdot r_d \cdot \frac{1}{G}$$

G is the shear modulus corresponding to the deformation level γ, which can be determined by an iteration process, knowing the value of the initial stiffness G0 (= ρVs^2), using the variation law G(γ)/G0 obtained by dynamic tests in laboratory.

1.9.6 Liquefaction

Liquefaction check

The phenomenon of liquefaction affects the saturated sand deposits which, in the course of a seismic event, or more generally during and immediately after a stress of cyclic type, undergo a drastic reduction of shearing resistance.
It is also now generally accepted that the main cause of liquefaction of saturated sandy soils, which occurs in the course of seismic events, is attributable to the increase in pore water pressures induced by cyclic shear stresses, which are due to the propagation of shear waves in the soil. The application of a succession of cyclic efforts in drained conditions initially generates a reduction of volume, however, if the stress occurs very rapidly compared to the drainage capacity of the deposit, it follows that the reduction of volume can not occur and the volume element will be subjected to an undrained loading process. The prevented volumetric strain will be accompanied by an increase in pore water pressure and a reduction of effective stresses, having to remain constant the total stresses. The increment of pore water pressure depends on the degree of densification of the soil and on the extent of the cyclic stress. If the ground is in a little dense state and the cyclic stress is sufficiently high, the increase in pore water pressure that follows can match the effective confinement stress and the soil particles are no longer subjected to any inter-granular stress.Under such conditions, and with no cohesion, the soil no longer has any resistance to shearing.



Sandy saturated soil before the effect of liquefaction (left figure), are seen the stresses that are exchanged (grains
represented by the red arrows). Saturated sandy soil in liquefaction (the inter-granular stresses are absent)

CASES WHERE LIQUEFACTION IS EXCLUDED (EC8)

The liquefaction check can be omitted when it occurs at least one of the following circumstances:

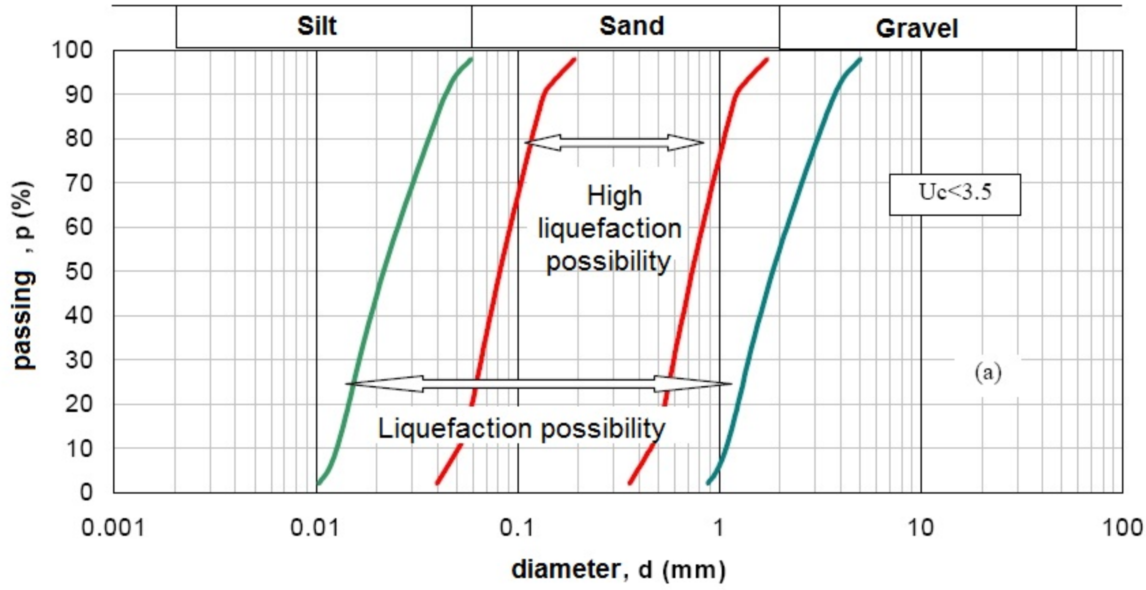
1. Expected seismic events with a magnitude M less than 5
2. Maximum expected acceleration at the surface less than 0.1 g
3. Maximum expected acceleration at the surface less than 0.15g and soils with characteristics that fall into one of three categories:

   - FC>20%, IP>10
   - FC≥ 35%, N'_{SPT} (corrected-normalized) >20
   - FC≤5%, N'_{SPT} (corrected-normalized) >25 ;

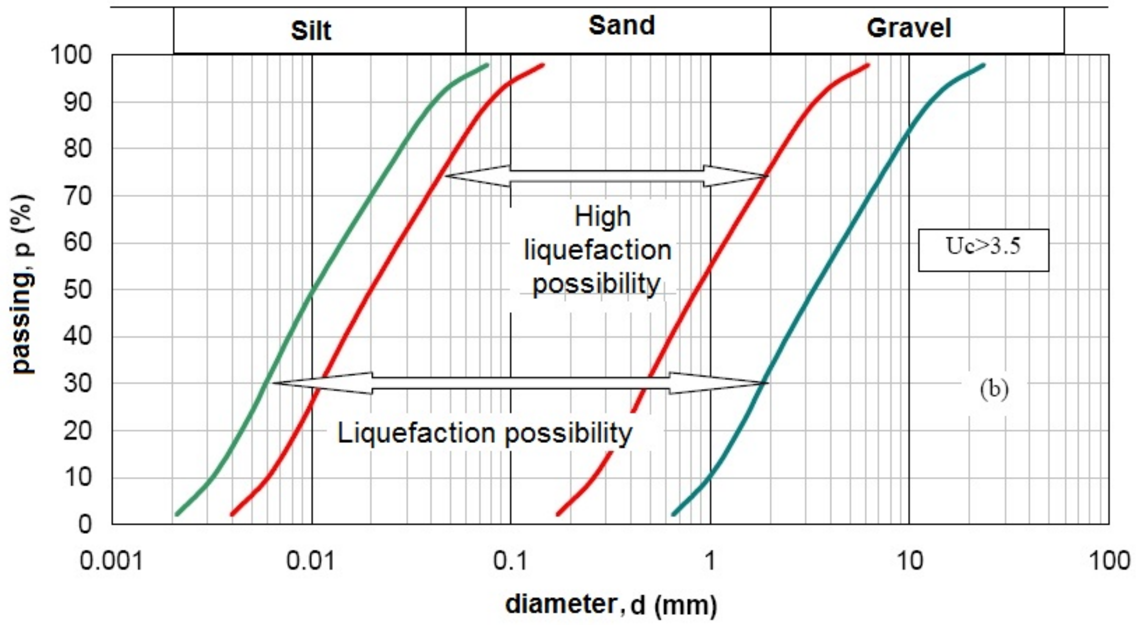
> **Note:**
>
> $$N'_{SPT} = N_{SPT} \cdot C_N \qquad C_N \left(\frac{p_a}{\sigma'_v} \right)^{0.5}$$
>
> where:
>
> pa is the atmospheric pressure
> σ'_v the effective vertical pressure

4. Material with Uc < 3.5 – Grain size of the soil outside the areas indicated in the image A
5. Material with Uc > 3.5 – Grain size of the soil outside the areas indicated in the image B

6. Seasonal average depth of the groundwater table greater than 15 m (as long as the ground surface is sub-horizontal and structures with shallow foundations).



A – Critical grain size bands Uc< 3.5



B – Critical grain size bands Uc>3.5

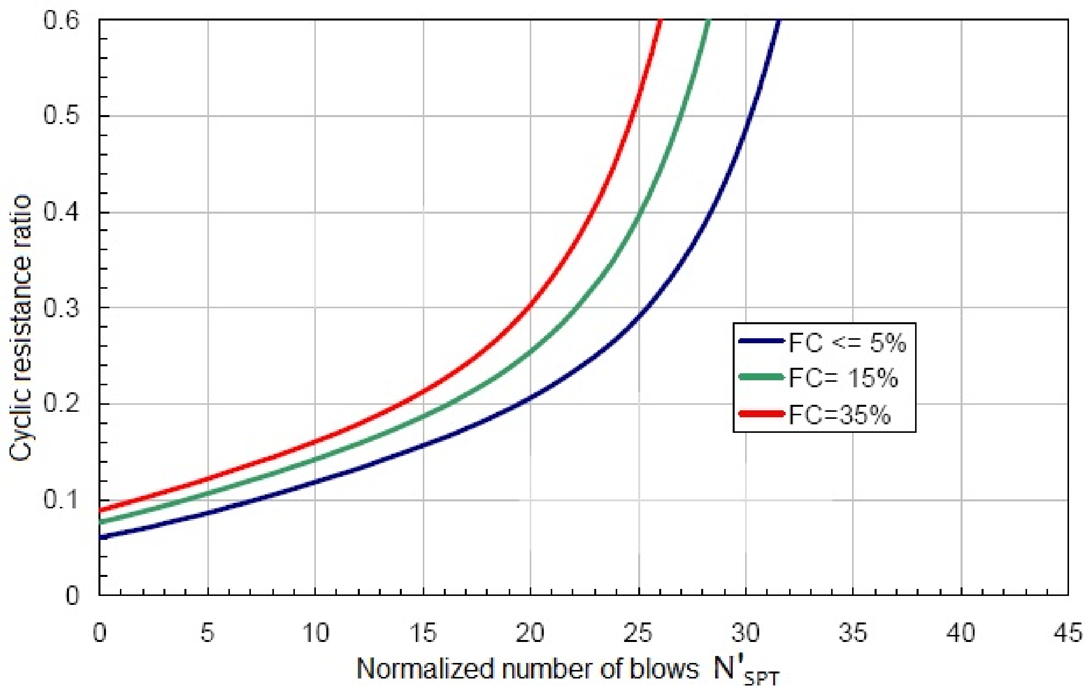
Seed and Idriss

The method used by LoadCap to estimate the liquefaction of a sandy saturated soil during a seismic event is that proposed by Seed and Idriss, the best known and most used of the simplified methods.

The method is based on the number of blows of the test Standard Penetration Test and only requires the knowledge of a few geotechnical parameters: the grain size, the relative density, the unit weight.
With this method, the liquefaction resistance factor FS is calculated by the ratio between the capacity of normalized resistance (R) and the application of cyclic resistance (T), multiplied by a scaling factor calculated considering an expected seismic event of a magnitude M = 6.5 which assumes a constant value equal to 1.19 (worst condition).
The capacity of normalized resistance with respect to the initial effective vertical pressure is expressed by the following relationship:

$$R = \frac{\tau_{ult}}{\sigma'_{v0}}$$

and can be determined from the graphic shown in the image below, as a function of the parameters derived from SPT suitably corrected and normalized.



Correlation between cyclic resistance capacity and corrected numbers of blows of a dynamic penetration test (N'_{SPT})

The application of cyclic resistance is expressed by the relationship:

where:

g acceleration of gravity;

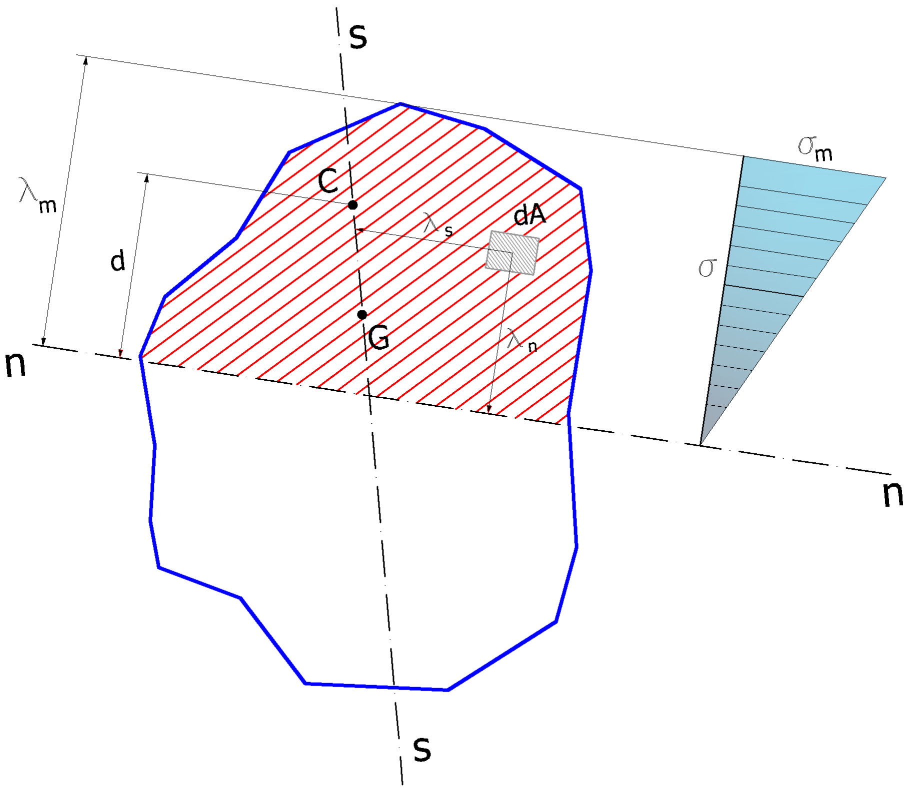
v 'v respectively the total vertical pressure and the effective pressure at the considered depth;

rd= 1-0.015z corrective coefficient that takes into account the deformability of the ground at the passing of the seismic shearing waves.

In the expression of the resistance application (T), to take account of the sporadic nature of the acceleration peaks, it is corrected the maximum cyclic stress induced by the earthquake by 35% obtaining a value of "equivalent uniform stress". If SF > 1.3 the soil is considered non-liquefiable.

1.9.7 Design stress

It is customary in the design practice to evaluate the pressures acting on the soil by adopting an approach "borrowed" from structural mechanics, considering the section sets of the foundation as presso-inflessa.
The calculation of the design pressure is performed considering the section partially reactant and with triangular distribution of the pressures on the soil.
The general case is that of a solid of De Saint Venant urged to normal eccentric stress, when the resultant of the external forces acting on the free base, is reduced to a normal stress **N** and a bending moment **M**.
This type of system is statically equivalent to a force **N** directed along the axis of the solid, applied at a point **C**, called the center of stress, different from the center of gravity **G** of the section partially reactant.
The joining **CG** provides the direction of the axis of the stress, together with that of the neutral axis defined as the anti polar of the center **C** of stress compared to the central inertia ellipse of the reactant section.



The unknowns of the problem, in the hypothesis of validity of the principle of conservation of plane sections and validity of Hooke's law, are three:

- two fix the position of the neutral axis
- another unknown variable is represented by the value of the stress in a generic point of the section

The solution of the problem is reached through a system of three equations:

1. Translation equilibrium equation in the direction normal to the section:

$$\int_{Ac} \sigma \cdot dA = N$$

2. Rotation equilibrium equation with respect to the neutral axis:

3. Rotation equilibrium equation with respect to the stress axis:

From the starting assumptions we can write the following relationship:

$$\sigma = \sigma_m \cdot \frac{\lambda_n}{\lambda_m}$$

Substituting the above equation into the three equilibrium equations, we obtain:

$$\frac{\sigma_m}{\lambda_m} \cdot \int_{Ac} \lambda_n \cdot dA = \frac{\sigma_m}{\lambda_m} \cdot S_n = N \qquad \Rightarrow \qquad \sigma_m = \frac{N}{S_n} \cdot \lambda_m$$

$$\frac{\sigma_m}{\lambda_m} \cdot \int_{Ac} \lambda_n^2 \cdot dA = \frac{\sigma_m}{\lambda_m} \cdot I_n = N \cdot d \qquad \Rightarrow \qquad \sigma_m = \frac{N \cdot d}{I_n} \cdot \lambda_m$$

having denoted by S_n the static moment of the reactant area with respect to the neutral axis and by In the moment of inertia of the reactant section with respect to the neutral axis or by the combination of the two results we obtain the position of the neutral axis:

$$d = \frac{I_n}{S_n}$$

Finally, from the rotation equilibrium equation with respect to the stress axis we obtain a relationship that expresses the condition that:
"the neutral axis and the stress axis are conjugated with respect to the inertia ellipse of the reacting section"

$$\int_{Ac} \lambda_n \cdot \lambda_m \cdot dA = 0$$

Known the position of the neutral axis we can calculate the stress at any point on the reacting section.

1.9.8 DESIGN OF SHALLOW FOUNDATIONS ACCORIDING TO: AASHTO – LRFD BRIDGE DESIGN SPECIFICATIONS – 9TH EDITION – 2020

The LRFD approach consists in checking the following equation:

$$Load(Q) \leq Resistance(R)$$

Where Q and R can represent forces, as well as stresses, deformations, displacements, etc. In this equation, Q represents an amplified value of actions, while R is a reduced value of resistances. The fundamental concept of LRFD design is expressed by the following relationship:

$$Q = \sum \eta_i \gamma_i Q_i \le \varphi R_n = R_r$$

In which

Q : factored load
Qi : force effect
ηi : load modifier
γi : load factor
Rr : factored resistance
Rn : nominal resistance (i.e., ultimate capacity)
φ : resistance factor

According to the LRFD approach, it is necessary to perform the verifications with reference to the following limit states:

- Strength limit state
- Service limit state
- Extreme event limit state
- Fatigue limit state

In geotechnical verifications, the 'fatigue limit state' does not need to be performed.

Limit State	Description
Strength	A design boundary condition considered to ensure that strength and stability are provided to resist specified load combinations, and avoid the total or partial collapse of the structure. Examples of Strength limit states in geotechnical engineering include bearing failure, sliding, and earth loadings for structural analysis.
Service	A design boundary condition for structure performance under intended service loads, and accounts for some acceptable measure of structure movement throughout the structure's performance life. Examples include vertical settlement of a foundation or lateral displacement of a retaining wall. Another example of a Service limit state condition is the rotation of a rocker bearing on an abutment caused by instability of the earth slope that supports the abutment.
Extreme Event (EE)	Evaluation of a structural member/component at this limit state considers a loading combination that represents an excessive or infrequent design boundary condition. Such conditions may include vessel impacts, vehicle impact, check flood (500-year flow event), and seismic events. Because the probability of these events occurring during the life of the structure is relatively small, a smaller margin of safety is appropriate when evaluating this limit state.

For each of these limit states, the standards identify various levels, which are indicated in the table below:

Load Combination Limit State	Load Combination Considerations
Strength I	Basic load combination relating to the normal vehicular use of the bridge without wind.
Strength II	Load combination relating to the use of the bridge by Owner-specified special design vehicles and/or evaluation permit vehicles, without wind.
Strength III	Load combination relating to the bridge exposed to wind velocity exceeding 55 mph without live loads.
Strength IV	Load combination relating to very high dead load to live load force effect ratios in the bridge substructures exceeding about 7.0 (e.g., for spans greater than 250 ft.).
Strength V	Load combination relating to normal vehicular use of the bridge with wind velocity of 55 mph.
Extreme Event I	Load combination including the effects of the design earthquakes. South Carolina uses 2 design earthquakes (SEE and FEE).
Extreme Event II	Load combination relating to collision by vessels and vehicles, check flood (500-year flow event), and certain hydraulic events.
Service I	Load combination relating to the normal operational use of the bridge with 55 mph wind

Most of these levels are to be performed only in the case of bridges subjected to specific wind conditions. The ones that need to be executed for all types of structures are those highlighted in red in the table (Strength I, Extreme I, Service I).

TYPES OF LOAD

In the LRDF approach, loads need to be amplified based on their type. Specifically, after distinguishing between permanent and transient loads, additional subcategories are identified, based on which the coefficient γi varies.

The permanent loads and their respective subcategories are as follows:
Permanent loads are present for the life of the structure and do not change over time. Permanent loads are generally very predictable. The following is a list of all loads identified by AASHTO LRFD Specifications as permanent loads:

- Force Effects Due to Creep – CR
- Dead Load of Components – DC
- Downdrag – DD
- Dead Load of Wearing Surface and Utilities – DW
- Horizontal Earth Pressures – EH
- Locked-In Erection Stresses – EL
- Vertical Earth Pressure – EV
- Earth Load Surcharge – ES
- Secondary Forces from Post-tensioning – PS
- Force Effects Due to Shrinkage – SH

The variable loads and their respective subcategories are as follows:

Transient loads may only be present for a short amount of time, may change direction, and are generally less predictable than permanent loads. Transient loads include the following:

- Blast Loading – BL
- Vehicular braking force – BR
- Vehicular centrifugal force – CE
- Vehicular collision force – CT
- Vessel collision force – CV
- Earthquake – EQ
- Friction – FR
- Ice load – IC
- Vehicular dynamic load allowance – IM
- Vehicular live load – LL
- Live load surcharge – LS
- Pedestrian live load – PL
- Settlement – SE
- Temperature gradient – TG
- Uniform temperature – TU
- Water load and stream pressure – WA
- Wind on live load – WL
- Wind load on structure – WS

The values of the coefficients γi to be used are reported in the following table, depending on the type of load and the considered limit state.

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I— *LL, IM & CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL, IM & CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

The coefficients indicated in this table with the symbol γp should be taken from the following tables:

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.5	0.9
○ Thermoplastic Culverts		1.3	0.9
○ All others		1.95	0.9
ES: Earth Surcharge		1.50	0.75

Table 3.4.1-3—Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effective}$	1.0	1.0
Steel Substructures	1.0	1.0

The γEQ and γse coefficients must be determined based on the specific project.

In the following it is indicated how to calculate the ηi coefficients, which depend on what value of γi is used. For loads for which a maximum value of γi is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \qquad (1.3.2.1\text{-}2)$$

η_i = load modifier: a factor relating to ductility, redundancy, and operational classification

η_D = a factor relating to ductility, as specified in Article 1.3.3

η_R = a factor relating to redundancy as specified in Article 1.3.4

η_I = a factor relating to operational classification as specified in Article 1.3.5

For the strength limit state:

$\eta_D \geq$ 1.05 for nonductile components and connections

= 1.00 for conventional designs and details complying with these Specifications

$\geq$ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by these Specifications

For all other limit states:

η_D = 1.00

For the strength limit state:

$\eta_R \geq$ 1.05 for nonredundant members

= 1.00 for conventional levels of redundancy, foundation elements where ϕ already accounts for redundancy as specified in Article 10.5

$\geq$ 0.95 for exceptional levels of redundancy beyond girder continuity and a torsionally-closed cross-section

For all other limit states:

η_R = 1.00

For the strength limit state:

$\eta_I \geq$ 1.05 for critical or essential bridges

= 1.00 for typical bridges

$\geq$ 0.95 for relatively less important bridges.

For all other limit states:

η_I = 1.00

BEARING CAPACITY CALCULATION

The ultimate value of the bearing capacity is calculated as follows:

$$q_R = \varphi_b q_n$$

where:
φb : resistance factor
qn : nominal bearing resistance

The nominal value is calculated using the well-known trinomial formula:

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$$

The values of the resistance factor are defined in the following table:

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

Alternatively, it is also possible to refer to the values indicated in the following table (valid for relative density values Dr > 35%).

<table>
<tr><th rowspan="3">Soil Friction Angle, ϕ'</th><th colspan="4">Loading Conditions</th></tr>
<tr><th rowspan="2">Vertical – Centric or Eccentric</th><th rowspan="2">Inclined - Centric</th><th colspan="2">Inclined - Eccentric</th></tr>
<tr><th>Positive</th><th>Negative</th></tr>
<tr><td>30° - 34°</td><td>0.40</td><td rowspan="3">0.40</td><td rowspan="2">0.35</td><td>0.65</td></tr>
<tr><td>35° - 36°</td><td>0.45</td><td rowspan="2">0.70</td></tr>
<tr><td>37° - 39°</td><td>0.50</td><td rowspan="2">0.40</td></tr>
<tr><td>40° - 44°</td><td>0.55</td><td>0.45</td><td rowspan="2">0.75</td></tr>
<tr><td>> 45°</td><td>0.65</td><td>0.50</td><td>0.45</td></tr>
</table>

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1.10 About general data

The project data must be changed from the "*General data*" menu:

1 Soil type
Chose between loose soil and rock, according to the type of soil the foundations rests upon. For foundations upon rock, the program automatically adjusts the data in the *"Soil stratigraphy"* window (ex. RQD).

2 Correction of parameters
For mainly sandy soils, Terzaghi suggested applying a correction to the geotechnical parameters, namely reducing the cohesion to 2/3 and the tangent of the shearing resistance angle to $0{,}67\cdot\tan(\varphi)$.

3 Foundation system data
Enter the geometrical data relevant to the foundations according to the instructions provided in the window for geometrical data input.
Amongst other geometrical dimensions, the depth of bearing surface D compared to the natural surface level as well as the foundation soil embedment are required: if you type in both and check the *"Embedded height = Bearing surface depth"* option, the program will consider the D depth in the calculation of the first term of the bearing capacity ($\gamma\cdot D\cdot Nq$). Otherwise, the program will assign to the D variable the Embedded height value. In the presence of plans of foundations fully or partially embeded, the excessive depth of the bearing surface can lead to high values of the bearing capacity due to the high value of the term ($\gamma\cdot D\cdot Nq$), therefore it can be useful to perform the computation with the embedded height, by clearing the option above, and enter the actual embedded part of the foundation in the ground.

4 Soil stratigraphy
The geotechnical data used by the program for the calculation of the bearing capacity and settlement must be entered in the window displayed when pressing the command *"Soil stratigraphy".*

Notes on geotechnical parameters

If ultimate limit state theory is used, the geotechnical parameters are taken as characteristic
Penetration soil tests
If are available the results of a dynamic penetration test in terms of N_{SPT} of the layer, can be performed a computation of the susceptibility to liquefaction of the layer in presence of seismic action, ground water table and cohesionless soil. This computation is made using the method of Seed and Idriss, and with the condition that the thickness of the layer is greater than 1 meter.

5 Loads

The input of the loads is only necessary in order to allow the calculation of the settlements. The input of a load for the evaluation of the bearing capacity of the soil is only used to determine the safety level as a Qlim/Qd bearing capacity – design load ratio. The program calculates different load conditions, both for the bearing capacity and for settlements, to be defined in the *"Loads"* window.
For each condition defined, the Type must be chosen: it will be meant as a Design type for the purposes of the assessment of the safety level on the soil bearing capacity; it will be meant as a Serviceability type for the purposes of the assessment of the settlements.Each load condition must be entered under the form of *"Design normal pressures"* or N normal stress, moments Mx and My and shears Hx and Hy. For instance, in the case of spread footings, the availability of such stresses is more immediate than the normal design pressure. In any case, the load entered refers to the foundation bearing surface and must therefore include the weight of the foundation as well. Besides, each load condition must be assigned already amplified of possible factors on the loads.

In order to define the safety levels acceptable by the user or imposed by applied regulations, it is necessary to insert in the Vertical and Horizontal Reduction Coefficients of the Bearing Capacity. In the same box (Earthquake + Partial coeff. soil geotechnical parameters + Resistances) are also defined the partial coefficients on the geotechnical properties of the soils (c’, cu, tanj, g): these coefficients represent the Mi partial coefficients introduced by Eurocodes, which reduce the geotechnical parameters defined in *"Soil stratigraphy"*. This type of coefficients is only considered for the load conditions belonging to Design type and not for those belonging to Serviceability type.
The seismic correction on the bearing capacity too is only referred to the load conditions concerning the bearing capacity and consequently belonging to the Design type. The values of the seismic reduction coefficients are described in the report produced in the text format *("Output" Menu > "Create report" command).*
The *"Generate combination" and "Assign loads"* buttons shown in the window activate the number and type of combination to be adopted according to the choice of the regulations to be implemented, respectively, and assign the normal design pressure an indicative value in case this datum is not available.

6 Distributed Loads

These are additional loads which can be assigned to the right or left sides of the foundations in order to take into account the presence of overloads adjoining the foundations (ex. bordering buildings). Their effect is only considered as an increase in the subsurface strain for the assessment of the settlements and in the interference of the bulbs.

7 Calculation methods

The analytic methods for the assessment of the soil bearing capacity are the classical ones present in the geotechnical literature: Terzaghi, Vesic, Meyerhof, Hansen and Brinch-Hansen, for soils; Terzaghi and Zienkiewicz, for rocks.

8 Calculation

The program includes calculation controls for the bearing capacity and the settlements.
Bearing capacity: The calculation of the bearing capacity gives the results of each Design load condition entered in the *"Loads"* window. The control proposes once more the same window as for loads with the addition of a results chart. The user can therefore make the necessary changes both in the loads and in the coefficients without exiting the control and entering again the "Loads" window from the "General data" menu.
For each load, the safety factor is returned as a Q_{lim}/Q_{ass} ultimate bearing capacity – assigned load ratio (design strain or pressure) and the Checked/Unchecked condition, according to whether the safety factor found is higher or not than the safety level imposed by the user in the *"Loads"* window.

Finally, for each author, the Winkler coefficient of subgrade reaction (ks) is calculated by means of the method proposed by Bowles:

$$ks = q_{lim}/\Delta H$$

with ΔH = 2,5 cm displacement considered as admissible.

1.11 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.: **Slope+Enter** to open GeoStru Slope software.
4) Quick access to GeoStru contacts.
Ex.: **Contact+?+Enter** to access the contact list.
5) Quick access to web features:

Ex.: www.geostru.com+Enter or geostru@geostru.com+Enter

2 Standards

2.1 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 unfavourable 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 gE and the shear resistance along the failure surface is divided by g 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	A2+**M2**+R1	**A1**+**R2**+M1	**A1***(A2^{+})+**M2**+R3
Slope	**A1**+M1+R1	A2+**M2**+R1	**E1**+**R2**+M1	E2+**M2**+R3
Piles and anchorages	**A1**+M1+R1	A2+M1+**R4**	**A1**+R2+M1	**A1***(A2^{+})+**M2**+R3

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 shearing 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 1 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 shearing 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_{γ}			1,0	
Resistance (R) (except for pile shaft in tension)		γ_R				1,0
Pile shaft 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 Rd 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 behaviour 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;

$$R_d = V'_d \tan \delta_d \ (6.3a)$$

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 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:

$$R_d = A_c \, c_{u;d} \ (6.4a)$$

or

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

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:

$$R_d \leq 0,4 \ V_d \ (6.5)$$

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:

- s 0 : 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;
- s 1 : settlement caused by consolidation;
- s 2 : 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.

2.2 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	Vs_{30} (m/s)	N_{SPT} (blows/30 cm)	cu (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-th 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 behaviour 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_I 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 ($a_g = \gamma_I \times a_{gR}$). (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 ($a_g = \gamma_I a_{gR}$);
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 $\eta = 1$ for 5% viscous damping, see (3) of this subclause.

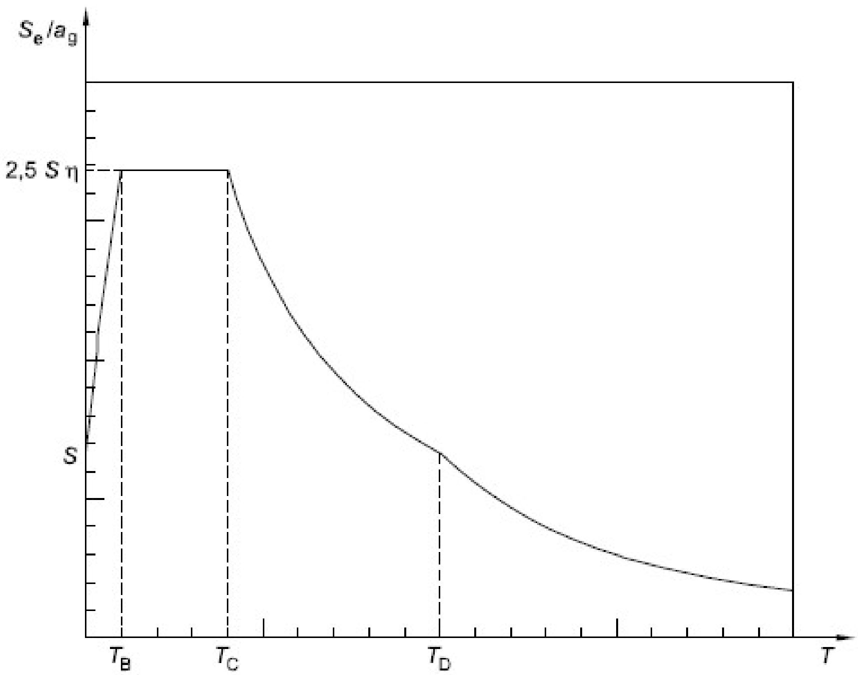

Figure 3.1 *- Shape of the elastic response spectrum*

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, 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*

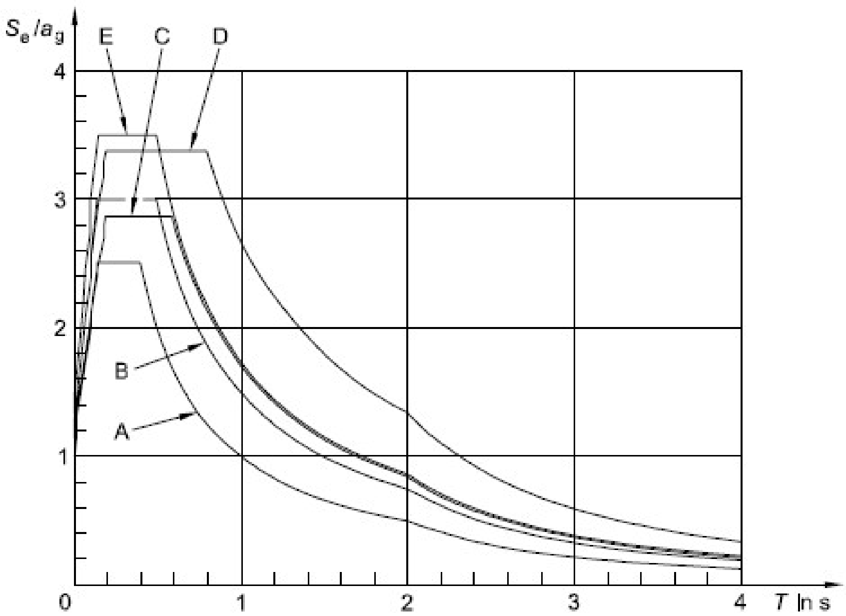

Figure 3.2 *- Recommended Type 1 elastic response spectra for ground types A to E (5% damping)*

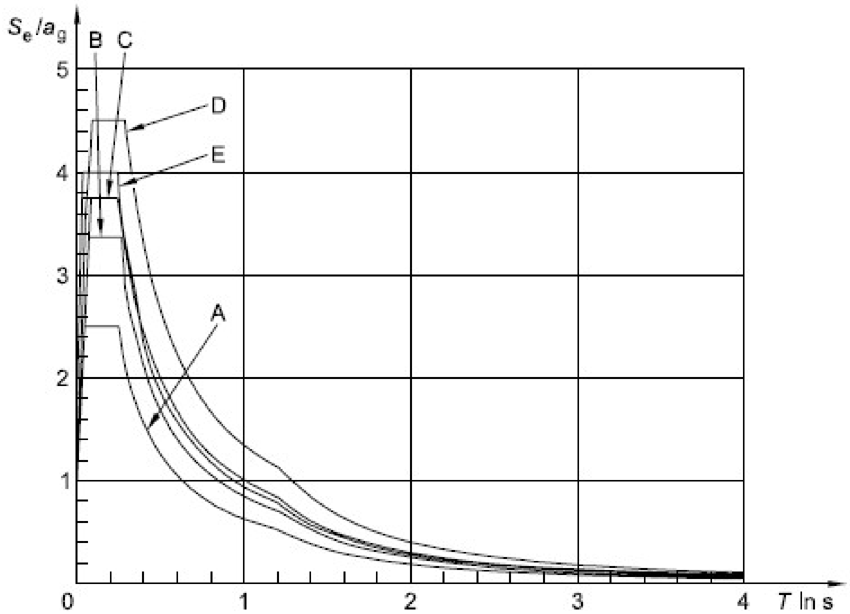

Figure 3.3 *- Recommended Type 2 elastic response spectra for ground types A to E (5% damping)*

Note 2: For ground types S1 and S2, 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:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \qquad (3.7)$$

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 behaviour 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 behaviour factor q.

3. The behaviour 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 behaviour 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 behaviour 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:

$$0 \le T \le T_B : S_{ve}(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2{,}5}{q} - \frac{2}{3} \right) \right] \tag{3.13}$$

$$T_B \le T \le T_C : S_d(T) = a_{vg} \cdot S \cdot \frac{2{,}5}{q} \tag{3.14}$$

$$T_C \le T \le T_D : S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2{,}5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases} \tag{3.15}$$

$$T_D \le T : S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2{,}5}{q} \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases} \tag{3.16}$$

where:

a_g, S, T_C e T_D	are as defined in 3.2.2.2;
$S_d(T)$	is the design spectrum;
q	is the behaviour factor;
β	is the lower bound factor for the horizontal design spectrum.

Note: The value to be ascribed to ß 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 behaviour 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 ag .
3. When site-specific data are not available, the minimum duration Ts 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 agS 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 agS 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:

(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 $\Psi_{2,i}$ 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 behaviour 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:

$$\mathbf{F_H = 0,5\ a\ SW}$$

$\mathbf{F_V = \pm\ 0,5\ F_H}$ if the ratio a_{vg}/a_g is greater than 0,6

$\mathbf{F_V = \pm\ 0,33\ F_H}$ if the ratio a_{vg}/a_g is not greater than 0,6.

Where:

α 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.

3 Utility

3.1 Conversion Tables

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

Converting slope inclination in degrees

From	To	Operation	Factor
N	kg	Divide by	9.8
kN	kg	Multiply by	102
kN	Tonn	Divide by	9.8
kg	N	Multiply by	9.8
kg	kN	Divide by	102
Tonn	kN	Multiply by	9.8

Forces conversion: 1 Newton (N) = 1/9.81 Kg = 0.102 Kg ; 1 kN = 1000 N

From	To	Operation	Factor
Tons/m^2	kg/cm^2	Divide by	10
kg/m^2	kg/cm^2	Divide by	10000
Pa	kg/cm^2	Divide by	98000
kPa	kg/cm^2	Divide by	98
Mpa	kg/cm^2	Multiply by	10.2
kPa	kg/m^2	Multiply by	102
Mpa	kg/m^2	Multiply by	102000

Pressures conversion: 1 Pascal (Pa) = 1 Newton/mq ; 1 kPa = 1000 Pa; 1 MPa = 1000000 Pa = 1000 kPa

3.2 Database of soil physical characteristics

Soil	Minimum value	Maximum value
Loose sand	0.48	1.60
Average compact sand	0.96	8.00
Compact sand	6.40	12.80
Average compact clayey sand	2.40	4.80
Average compact silty sand	2.40	4.80
Compact sand and gravel	10.00	30.00
Calyey soil with qu< 2 Kg/cm²	1.20	2.40
Calyey soil with 2< qu< 4 Kg/cm²	2.20	4.80
Calyey soil with qu> 2 Kg/cm²	>4.80	

Approximate values of Winkler's constant K in Kg/cm^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 volume weight in Kg/cm^3

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 the friction angle φ, in degrees, for soils

Soil	Value
Sandy clay	0.20
Soft clay	0.10
Plastic clay	0.25
Semisolid clay	0.50
Solid clay	1
Tenacious clay	2÷10
Compact silt	0.10

Approximate values of cohesion 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 module, in Kg/cm^2, 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

Soil	Maximum value of n	Minimum value of n
Ice	0.36	
Concrete	0.15	

Approximate values of the Poisson's ratio for soils

Rock	Minimum value	Maximum value
Pumice	500	1100
Volcanic tuff	1100	1750
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

Approximate values of specific weight for some rocks in Kg/m^3

Rock	Minimum value	Maximum value
Granite	45	60
Dolerite	55	60
Basalt	50	55
Sandstone	35	50
Calyey schist	15	30
Limestone	35	50
Quartzite	50	60
Marble	35	50

Approximate values of the friction angle φ, in degrees, for rocks

Rock	E		n	
	Maximum value	Minimum value	Maximum value	Minimum value
Basalt	1071000	178500	0.32	0.27
Granite	856800	142800	0.30	0.26
Crystalline schist	856800	71400	0.22	0.18
Limestone	1071000	214200	0.45	0.24
Porous limestone	856800	35700	0.45	0.35
Sandstone	428400	35700	0.45	0.20
Calyey schist	214200	35700	0.45	0.25
Concrete	Variable		0.15	

Approximate values of the elastic module and Poisson's ratio for rocks