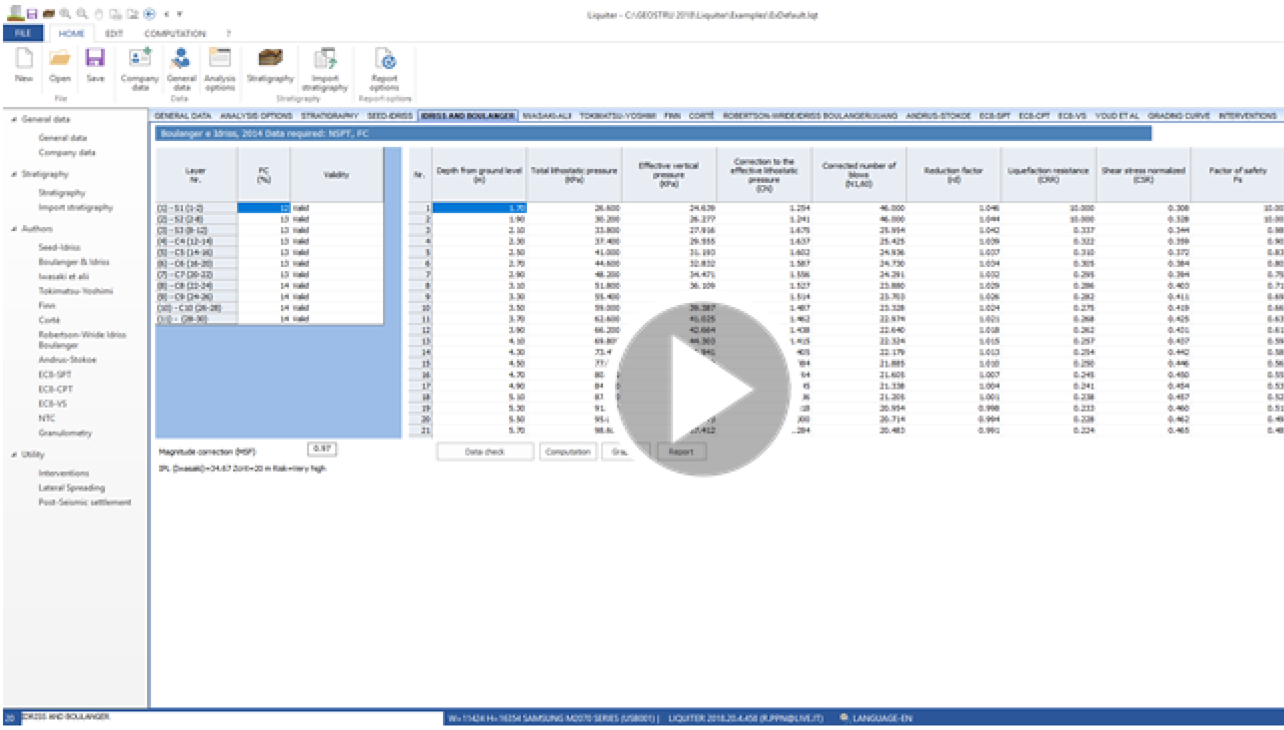
Liquiter

1 Liquiter

LIQUITER software is designed for soil liquefaction analysis and supports a wide variety of field tests. The results of the analysis are presented as:

- Factor of Safety against Liquefaction / Liquefaction Potential Index
- Cyclic mobility of clay
- Liquefaction of sand and clay
- CSR, variable CSR with depth from SHAKE results
- Reconsolidation Settlement, Lateral Spreading
- Residual Strength.



Liquiter supports the following field tests for soil liquefaction analysis:

- Standard Penetration Test (SPT)
- Tests that calculate the shear waves velocities (Vs)
- Cone penetration test (CPT)
- Cone penetration test electric (CPTE)
- Cone penetration test Piezocone (CPTU)
- Grading curve

Magnitude Scaling Factor (MSF)

- Seed & Idriss,1990
- Idriss, 1995

- Andrus & Stokoe, 1997, 2000
- Idriss & Boulanger, 2008, 2014

Stress Reduction Factor (Kσ)

- NCEER (Youd, 1997)

Shear Wave Velocity Normalization with Depth

- Robertson et al., 1992

SPT Correction (Cn)

- Liao and Whitman, 1986
- Idriss & Boulanger 2014

Post Liquefaction Residual Strength (Sr)

- Idriss & Boulanger, 2009, 2014

Fines Content Correction

- Idriss & Seed, 1997 (NCEER Workshop)
- Robertson & Wride, 1997 (NCEER Workshop)
- Idriss & Boulanger, 2014

Calculation of liquefaction safety factor using:

- Andrus-Stokoe
- Boulanger-Idriss-CPT-2008
- Boulanger-Idriss-CPT-2014
- Boulanger-Idriss-NSPT, 2014
- Corte
- Eurocode-8
- Finn
- Iwasaki
- Robertson-Wride
- Seed-Idriss
- Tokimatsu-Yoshimi
- Youd et. al., 2001
- C. Hsein Juang 2006

Lateral Spreading

- Youd et. al., 2002
- Barlett and Youd, 1995

Liquefaction potential index (LPI)

- Iwasaki et al.
- Somez (2003)

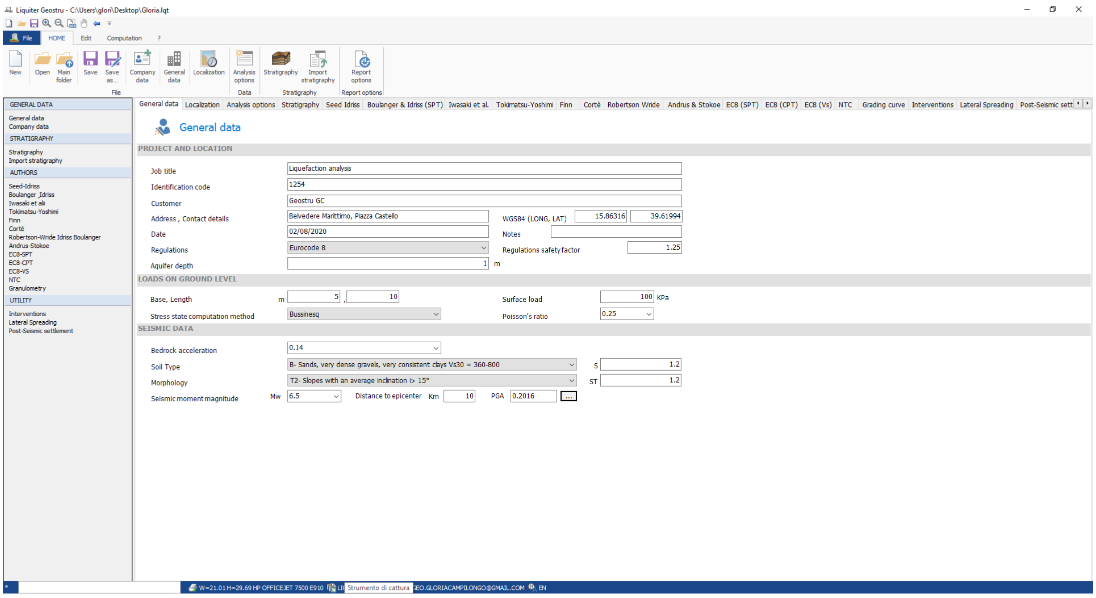
2 Project

2.1 Creation of a new project

How to create a new project

The Liquiter software (as well as the new versions of the Geostru software) is created in a easy and intuitive way. Therefore, in this section, it will be described only a few steps for creating a new project.

To create a new project, click on the "File 3" menu and then "New", but the software allows to speed up the steps by making the main window relating to "General Data" in the general interface. Within it it will be possible to enter the project data and other data useful for the purposes of the calculation.



Proceed by clicking on the other windows on the menu bar and / or on the side (vertical bar on the left).

3 Menu guide

3.1 File

New (Ctrl+N)

Create a new project. The command is also available on the Standard tool bar.

Open

An existing project, saved in (*.lqf) format, can be opened. The command is also available on the Standard tool bar.

Save (Ctrl+F12)

Saves data inserted in the current project. The command is also available on the Standard tool bar.

Save as

Saves the project under a different name.

Exit

Allows to exit the current project

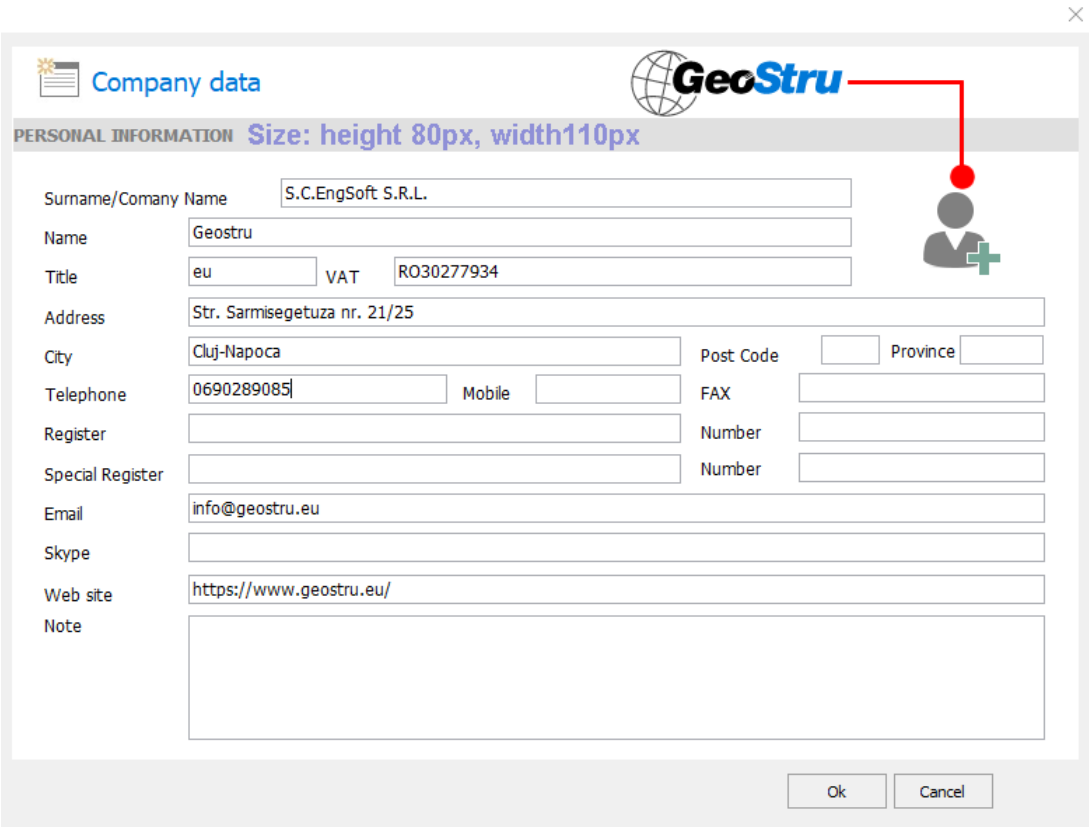
Recent projects

List of recent projects.

3.2 Data

Company data

The command opens a window in which you can inser personal or company data. A range of information can be inserted: company name, title, VAT number, address, etc, data then will be shown in the computation report.
Clicking on the add an image / logo (*. bmp, *. jpg) can be inserted.



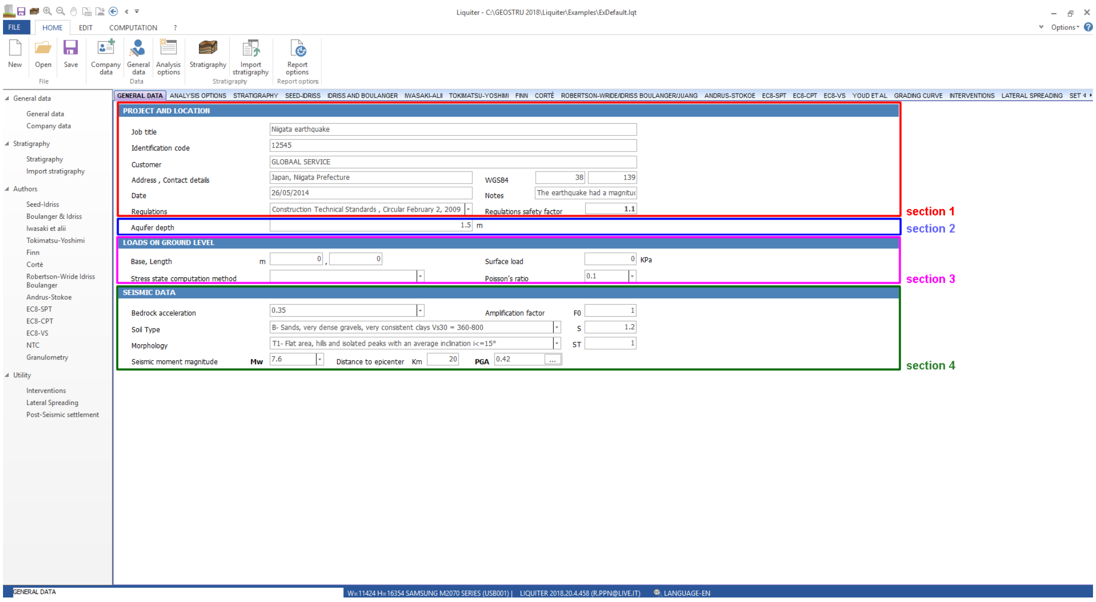
General data

In *section 1 (Project and location)* can be inserted information regarding the project and location of the work. Data can then be used in the graphical output. In the same section must be inserted the normative to consider and the safety factor.

In *section 2 (Loads in ground level)* the software gives the possibility to calculate the stress increment in the ground induced by a load on the surface. In *section 3* define the geometry of the load, choose the computation methodand insert the required elastic parameter.

To calculate the maximum shear stress induced by the earthquake the software uses simplified methods developed by various authors, while for the computation of the safety factor is required the **maximum acceleration induced by the earthquake in surface**. For this purpose, in *section 4*, insert the required seismic data useful to determine this parameter.

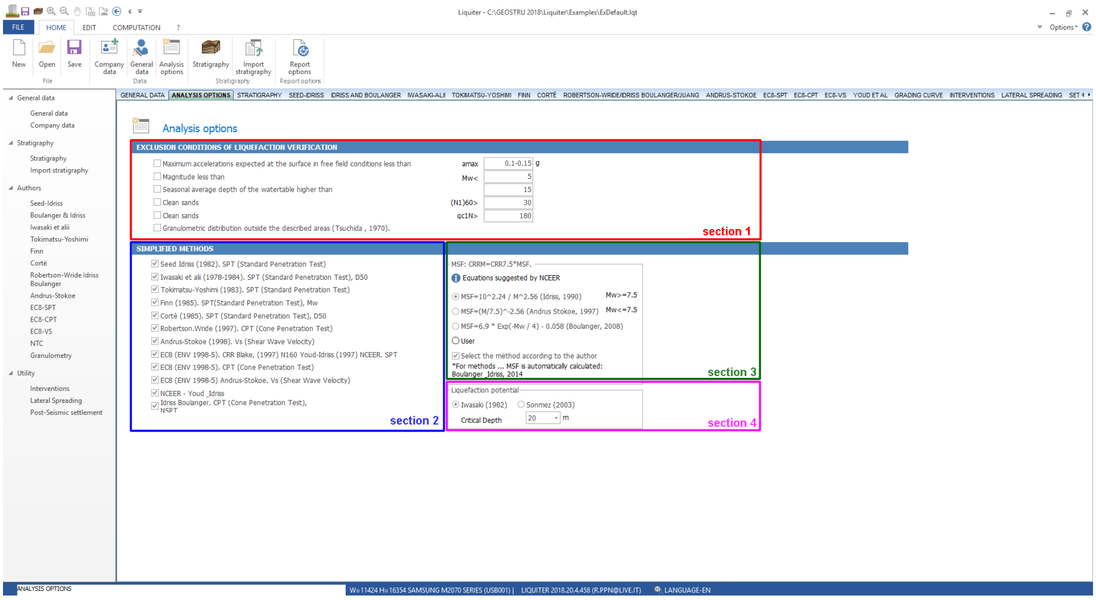
The magnitude and **distance to epicenter** is used by the computation code to **correct** the **Cyclic Stress Ratio** using correlations proposed by various authors *(to choose in the analysis options).*



Analysis options

Using this window the user can define the exclusion conditions for the liquefaction verification based on the chosen normative. The limits can be altered manually, inserting the values in the appropriate fields (*section 1*).

The methods to be used can be chosen in *section 2.*
To adjust the value of **CSR** (*Cyclic Stress Ratio)* to earthquakes having a magnitude different from 7.5 it must be inserted the **MSF** (*Magnitude Scaling Factor).* The computation equations suggested by NCEER in base of the chosen author must be selected in *section 3.*Il programma fornisce anche la stima dell'indice del potenziale di liquefazione, Iwasaki(1982) e Sonmez (2003) - *sezione 4,* riferendosi ad una profondità critica di 10 oppure 20 metri.



3.3 Stratigraphy

Import stratigraphy

The command opens a window that allows to choose the file to be imported (file type *.txt, *.edp). The files imported from other GeoSTru programs (like Static Probing, Dynamic Probing, etc.) contain all information required in the stratigraphy table and used in the computation.

Stratigraphy

Insert in the table the required data:

DB

Click on this column to choose a type of soil from the predefined database. The user can customize the database by adding, modifying or deleting soil types.

Description

Insert a description for the layer.

Layer elevations

Enter the initial and final elevation of the layers (ground level is considered elevation 0).

The initial quota of layer 1 must be set equal to zero!

Natural/saturated unit weight
Enter the unit weights for the layer.

Average blow number (NSPT)
Assign the average number of blows from the SPT test.

D50 granules (mm)
Enter the value of the diameter corresponding to 50% passing (grading curve).

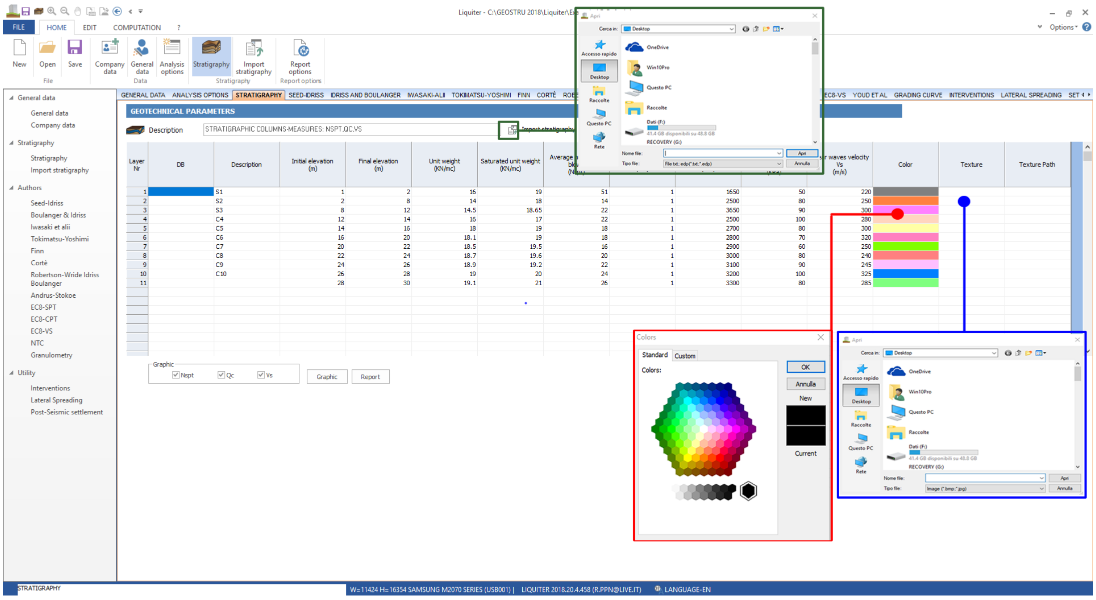
Resistance qc and resistance to side friction fs
Enter point resistance and side resistance values derived from the static penetration test.

Shear waves velocity Vs
Insert the shear wave velocity for the layer derived form the penetration soil tests.

Color
Click on the cell - the color selection window opens and the user can choose a color to assign to the layer.

Texture
Clicking the cell you want to enter the texture it opens a window that allows to choose the file to be imported as texture *(image file in *.jpg, *.bmp formats)* and that will be assigned top the layer in the graphics.

The ***"Graphic"*** button generates the *stratigraphy-depth* graphic for the chosen parameters (*NSPT, Qc, Vs*).
Clicking on "*Report*" button in the lower side of the page, the software offers the possibility to print or export the stratigraphy table.



3.4 Graphic options

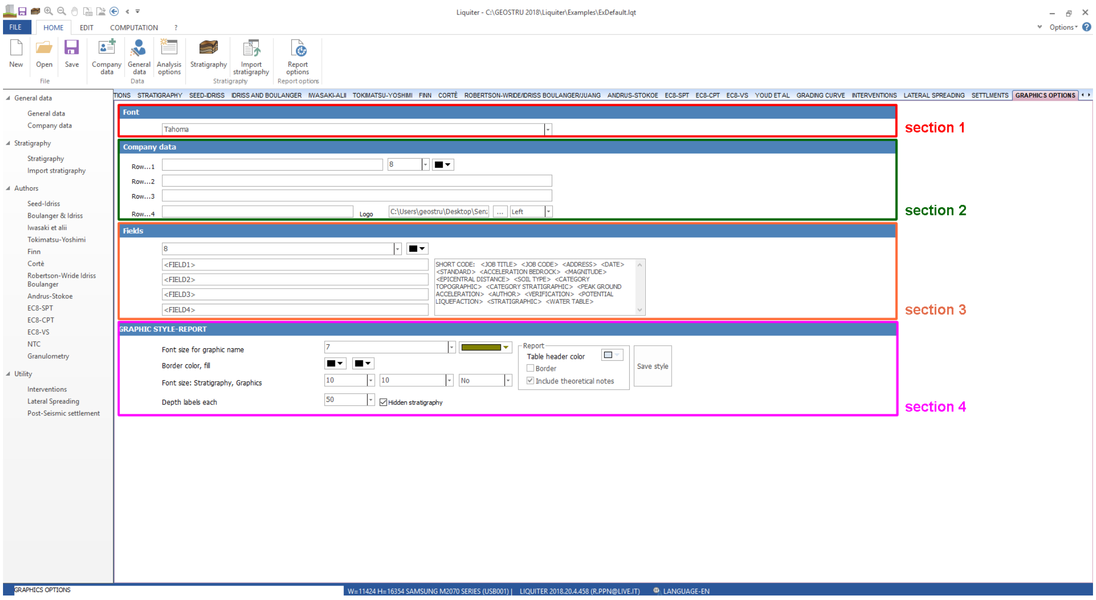
From *section 1 (Font)* it can be chosen the font to be used for the texts in the graphics.

Company data can be entered in *section 2.* In the same section can be chosen the color and size of the font and entered the logo *(*.bmp, or *.jpg image file)* and decide its position *(left or right).*

In *section 3* can be entered additional fields, if the users decides to copy/paste the **short codes** available in the fields *Field1, Field2, Field3, Field3* it can create a style, save it (using the save button in the bottom of the window) and use it for other projects as well. Text color and size can also be customized.

The size and color of the characters of table headers, table border color, table filling, size and color of stratigraphy graphic and other graphics can be assigned in *section 4*.

From *section 5* the user can define the colors of the table header, choose weather to include or not the theoretical notes in the reports and save the style for further projects.



3.5 View

Use the zoom options to increase or decrease the size of the image displayed in the drawing area . The zoom options can be activated from the Standard toolbar, or with a right click on the drawing area.

Zoom (+)

The command applies a magnification factor to display a more detailed image.

Zoom (-)

Using this command it is applied a reduction factor to view a larger part of the page with smaller sizes.

Zoom (100%)

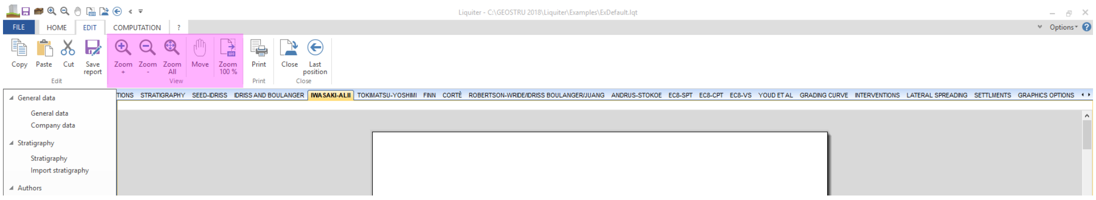
View drawing at 100% of normal size.

Zoom (all)

Shows the full view of the project work inside the drawing area.

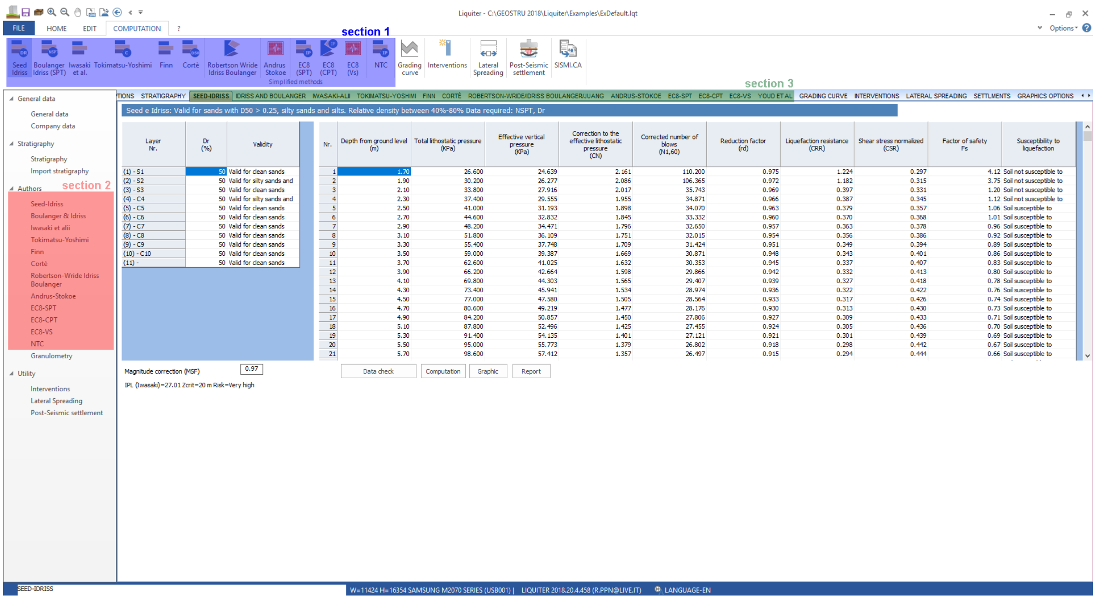
Move

The command is activated using the appropriate command in the ("Hand") or by holding the left click on the drawing and moving it in any direction: horizontally, vertically or diagonally. The magnification of the drawing remains the same as well as its orientation in space. The only change is the portion of the drawing displayed.



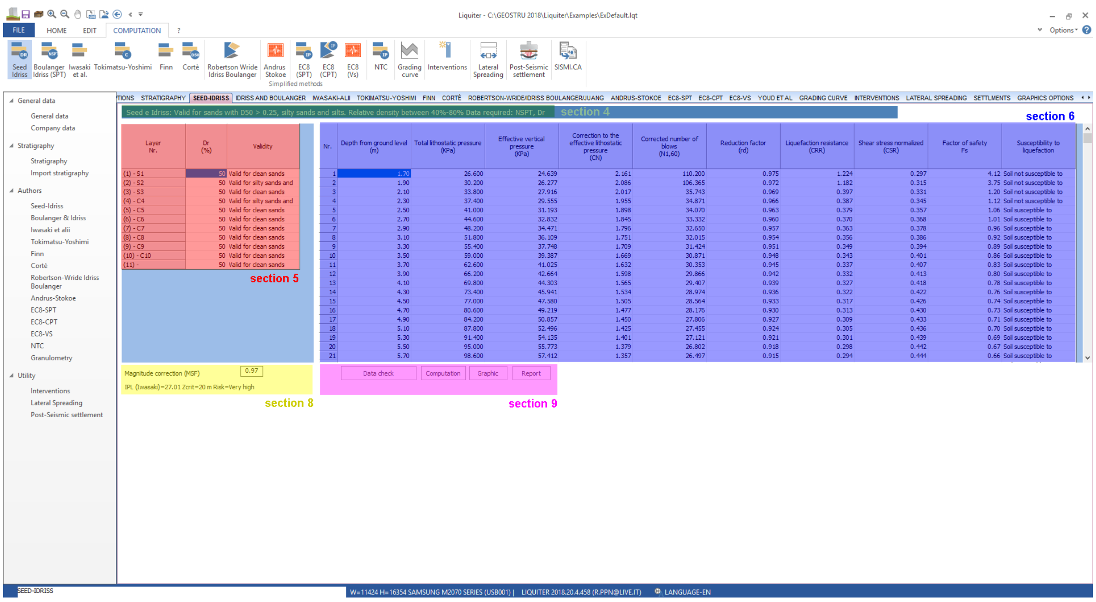
3.6 Computation

From this section the user can choose all the methods implemented in the program for the liquefaction check.
The choice can be made from the controls placed on the standard toolbar (*section 1*), from the panel located to the left side of the work area (*section 2*) or using the commands in *section 3*.



Let's choose an example: method **Seed Idriss**
in section 4 are shown the validity conditions and the data required to perform the analysis, in this case: The method is valid for sands with D50> 0.25, silty sands and silts. Relative density between 40-80%, the data required for processing are NSPT and the D50 to be assigned in *Stratigraphy* 7.
After assigning the stratigraphy it must be defined, in the table from *section 5,* the validity options of the method layer by layer. The program processes the data and returns the results in the table in *section 6*

providing the results of the analysis. The results reported in this table can be exported using the copy/paste command.
For earthquakes with a magnitude greater than 7.5 is calculated the value of the correction factor on the magnitude MSF using the correlation or the method chosen in the Analysis options 4. The calculated value is shown on screen as well as the index of liquefaction potential (*IPL*). The chosen computation method: Idriss or Sonmez, assigned value to critical depth z_{crit}:10 or 20 m and the risk associated with it (*section 7*).
The program offers an accurate computation report, including the theoretical references and organizing the analysis results in tabular form.
After processing the data, it can be generated a graphic that, in the case of Seed and Idriss, is structured as follows: (*stratigraphic column-depth, NSPT-depth, FS-depth*).
To generate the output just select the specific commands (*section 8*), the report and the graphic options can be changed from the Report options 9.



__N.B.:__ The options for the computation of the liquefaction potential can be found in the section Analysis options.

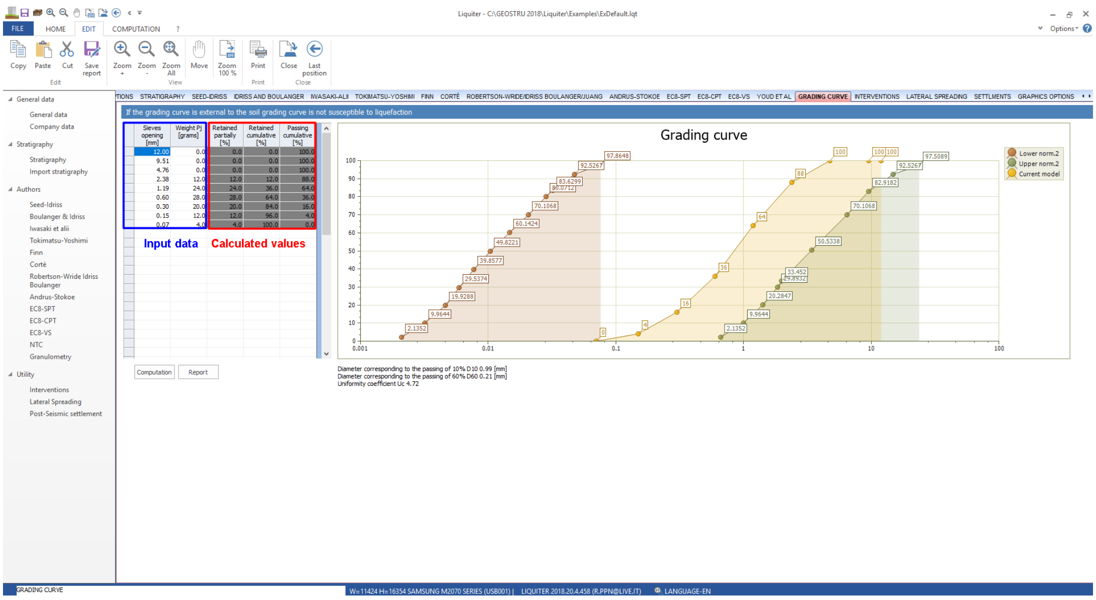
Grading curve

If the granulometric distribution of the soil is external to areas indicated as "critical", the liquefaction verification can be excluded, the soil is not susceptible to liquefaction.

The command opens a window consisting of a table and a graphic. In the table must be inserted:

- Sieves opening (the ones from the A.S.T.M. series, in descending order, are: 38.10 mm, 25.40 mm, 19.00 mm, 12.70 mm, 9.51 mm, 4.76 mm, 2.38 mm, 1.19 mm, 0.595 mm, 0.297 mm, 0.149 mm, 0.074 mm.)
- Weight Pj - the weight of the grains retained partially. The software calculates the grains retained partially (%), cumulated retained grains (%) and cumulated passing grains (%). While this data is entered the software plots the grading curve of the current soil model, together with the lower and upper grading curves indicated by the normative (lower norm, upper norm.) for uniformity coefficient Uc< 3.5 and for Uc > 3.5

By clicking on the ***"Report"*** button the graphic and the table are prepared for print.
The report can be saved in *. bmp format using the command "*Save as*"on the standard toolbar (or right click on the graphic).



4 Theoretical notes

4.1 Simplifid methods

4.1.1 Introduction

The simplified methods are based on the relationship between the shear stresses which produce liquefaction and those induced by the earthquake; therefore they need to evaluate the parameters for both seismic event and deposit. The resistance to liquefaction of the deposit is then calculated in terms of liquefaction resistance factor.

$$FS = \frac{CRR}{CSR}$$

where **CRR** *(Cyclic Resistance Ratio)* indicates the resistance of the soil to cyclic shear stresses and **CSR** *(Cyclic Stress Ratio)* the maximum shear stress induced by the earthquake.
The proposed simplified methods are different especially as it concerns the calculation of CRR, the liquefaction resistance. The most used parameter is the blow count from the SPT even if nowadays is preferred the computation of the liquefaction potential from CPT test or measurements of the shear waves velocities Vs. These methods are generally used for the design of structures with average importance.
Seed and Idriss (1971-1982), propose a simple procedure based on the assumption of homogeneous soil. Assuming the vertical propagation of seismic share waves, a soil column of height z (Figure 1) moves rigidly in the horizontal direction and therefore the maximum shear stress at the depth z is given by:

$$\tau_{max} = \frac{a_{max}}{g} \cdot \gamma \cdot z$$

where $\mathbf{a_{max}}$ is the maximum acceleration at the surface, **g** the acceleration of gravity and γ is the dry unit weight of soil.
Since in reality the soil is deformable, the shear stress is less than that in the case of a rigid body, so we have to introduce a reduction factor **rd**. Normalizing with the vertical effective pressure and referring to an average value τ_{av} rather than to a maximum value τ_{max} we obtain:

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) \cdot r_d$$

The value **rd** is the stress reduction coefficient and is determined as follows (**Liao e Whitman**,1986):

$$r_d = 1.0 - 0.00765z \; per \; z \leq 9.15m$$

$$r_d = 1.174 - 0.0267z \; per \; 9.15m < z \leq 23m$$

Where **z** is depth below ground surface in meters.

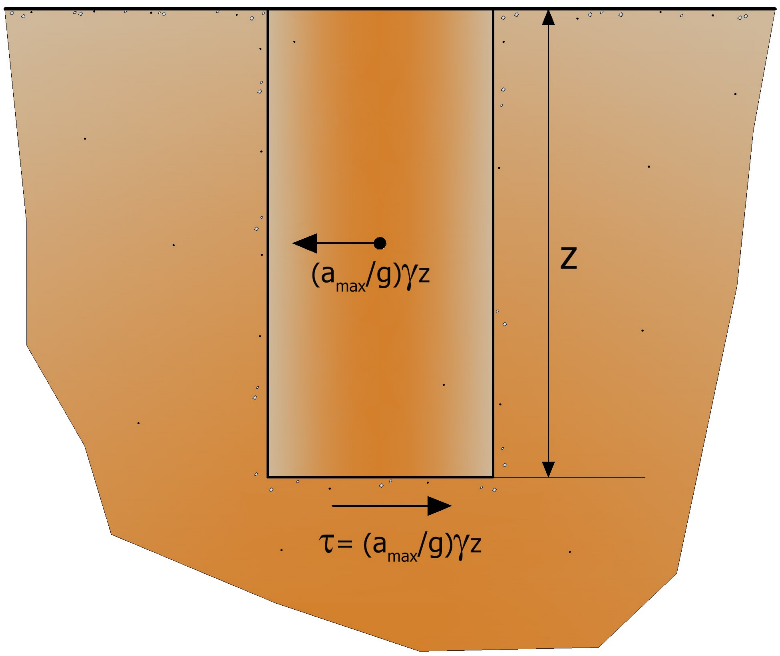


***Figure 1** – Shear stress induced by the earthquake to depth "z"*

4.1.2 Seed and Idriss

The method of **Seed e Idriss (1982)** calculates the CSR using the following formula:

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) \cdot r_d$$

To determine the value of the reduction factor **r_d** is used the formula proposed by (**Liao e Whitman**,1986):

$$r_d = 1.0 - 0.00765z \; per \; z \leq 9.15m$$

$$r_d = 1.174 - 0.0267z \; per \; 9.15m < z \leq 23m$$

For Magnitude 7,5 Earthquakes the original curve of Seed and Idriss (1982) is considered whose formula is:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200}$$

This equation is valid for N1(60)<30. For N1(60)≥30, clean granular soils are too dense to liquefy and are classed as “non liquefiable”.

For different Magnitude (greater or less than 7.5) Seed e Idris (1982) intoduced the Magnitudo Scaling Factor MSF defined by following equation:

$$MSF = \frac{10^{2.24}}{M_w^{2.56}}$$

while for the correction factor **MSF** refers to the values reported in Table 1 obtained by several researchers, including **Seed H. B. and Idriss I. M** (1982).

Table 1- *Scale factor of the magnitude derived from several researchers*

Magnitude	***Seed H.B. & Idriss I.M.*** *(1982)*	***Ambraseys N.N*** *(1988)*	***NCEER*** *(Seed R. B. et al) (1997; 2003)*
5,5	*1,43*	*2,86*	*2,21*
6,0	*1,32*	*2,20*	*1,77*
6,5	*1,19*	*1,69*	*1,44*
7,0	*1,08*	*1,30*	*1,19*
7,5	*1,00*	*1,00*	*1,00*
8,0	*0,94*	*0,67*	*0,84*
8,5	*0,89*	*0,44*	*0,73*

The Cyclic Resistance Ratio is calculated as a function of magnitude, the number of blows in the SPT test, the effective vertical pressure and the relative density.

Initially is calculated the corrected number of blows to the desired depth to take account of the lithostatic pressure by the following expression:

$$(N1)60CS = \alpha + \beta (N1)60$$

where α e β are coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for FC} \leq 5\%$$

$$\alpha = \exp[1.76 - (190/FC^2)] \quad \text{for } 5\% < FC < 35\%$$

$$\alpha = 5.0 \quad \text{for FC} \geq 35\%$$

$$\beta = 1.0 \quad \text{for FC} \leq 5\%$$

$$\beta = [0.99 + (FC^{1.5}/1{,}000)] \quad \text{for } 5\% < FC < 35\%$$

$$\beta = 1.2 \quad \text{for FC} \geq 35\%$$

Other corrections that influence SPT results are incorporated in these corrections:

(N1)60 =NmCNCECBCRCS

where Nm is the measured standard penetration resistance; CN is a correction factor to normalize Nm to a common reference effective overburden stress; CE is the corretion for hammer energy ratio (ER); CB is the correction factor for borehole diameter; CR is the correction factor for rod length; and CS is the correction for samples with or without liners.

C_N is determined by the relation:

$$C_N = \left(\frac{Pa}{\sigma'_{v0}} \right)^n$$

where σ'_{vo} is the effective pressure, **Pa** the atmospheric pressure (~ 100 kPa) expressed in the same units as σ'_{vo} and **n** an exponent whose value is 0.5 and depends on the relative density of the ground (Figure 1).

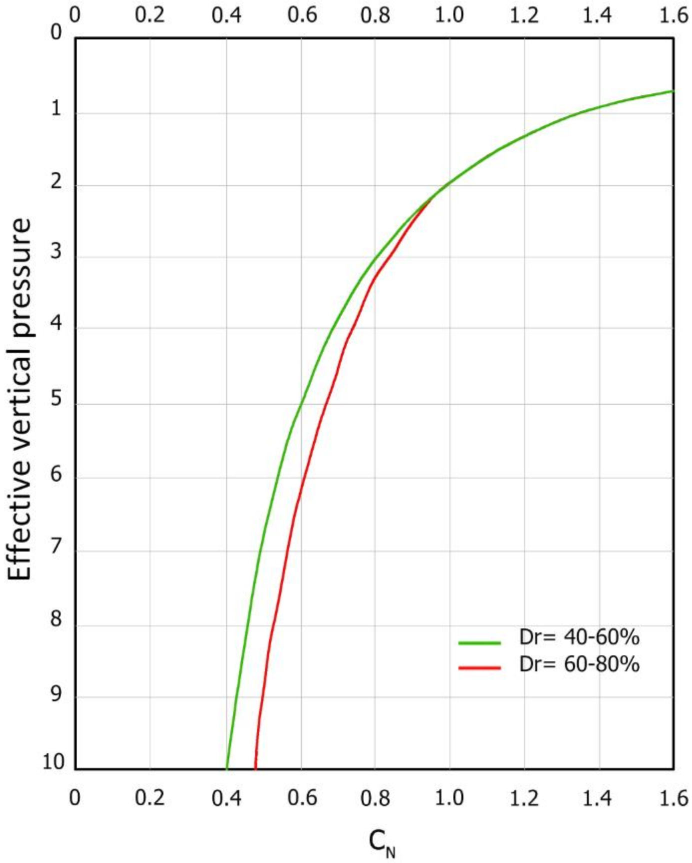


Figure 1 *– Correction coefficient CN*

To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (**FS**) against liquefaction is written in terms of CRR, CSR, and MSF as:

FS= (CRR7.5/CSR)MSF

where CSR is the calculated cyclic stress ratio generated by the earthquake shaking; and CRR7.5 is the cyclic resistance ratio for magnitude 7.5 earthquakes.

4.1.3 Iwasaki et al.

This method was developed based on the observation that the severity of the damage produced by liquefaction on structures is related to the volume of soil liquefied inside the deposit.
The method is based on the estimation of two quantities: **the factor of safety (FS)** and the **liquefaction potential index (LPI)**. The liquefaction potential index LPI, indicative of the extension that the

phenomenon of liquefaction can have within the stratum, is derived from the expression:

$$LPI = \int_0^{20} F(z) \cdot W(z) dz$$

where:

$$F = 1 - F_S \qquad \text{for } F_S \le 1$$
$$F = 0 \qquad \text{for } F_S > 1$$
$$W(z) = 10 - 0.5 \cdot z$$

CSR for earthquakes with a magnitude greater than 7.5 is calculated using the method proposed by Seed and Idriss (1982).

For the computation of the Cyclic Resistance Ratio **CRR** are proposed the following expressions obtained from numerous tests of undrained cyclic strength:

- for soils with 0,04 mm ≤ D_{50} ≤ 0,6:

$$CRR = 0.0882\sqrt{\frac{N_m}{\sigma'_{v0} + 0.7}} + 0.225 \log_{10}\left(\frac{0.35}{D_{50}}\right)$$

- for soils with 0,6 mm ≤ D_{50} ≤ 1,5:

$$CRR = 0.0882\sqrt{\frac{N_m}{\sigma'_{v0} + 0.7}} - 0.05$$

where:
D_{50} is the diameter of granules to 50% (in mm) and $\mathbf{N_m}$ is the average number of blows in the standard penetration test SPT.
The classification of the liquefaction risk through the method of **Iwasaki et al.** is shown in Table 2.

Table 2 - *Risk classes*

LPI	*Liquefaction risk*
LPI = 0	*Very low*
0 < LPI ≤ 5	*Low*
5 < LPI ≤ 15	*High*
15 < LPI	*Very high*

4.1.4 Tokimatsu and Yoshimi

To take into account the magnitude of the earthquake, the method proposed by **Yoshimi and Tokimatsu** calculates the Cyclic Stress Ratio with the following equation:

$$CSR = 0.65 \frac{a_{\max}}{g} \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot r_d \cdot r_n$$

where is introduced a correction factor $\mathbf{r_n}$ function of the magnitude **M**:

$$r_n = 0.1 \cdot (M - 1)$$

The liquefaction resistance is obtained by the following expression:

$$CRR = a \cdot C_r \left[\frac{16\sqrt{N_{1.60} + \Delta N_f}}{100} + \left(\frac{16\sqrt{N_{1.60} + \Delta N_f}}{C_s} \right) \right]$$

where:

a = 0,45.
C_r = 0,57.
n = 14.
ΔN_f = 0 for clean sands and ΔN_f = 5 for silty sands
$N_{1,60} = [1,7 / (\sigma'_{v0} + 0,7)]\ N_m$
Cs is an empirical constant which depends on the amplitude of the shear deformation.

The previous relationship was obtained by the authors by correlating the results obtained from cyclic triaxial tests with the results of standard penetration tests SPT.
The authors, for design purposes, suggest to adopt a value of **FS > 1.5** for loose and medium sands and **FS > 1.3** for medium-dense sands.

4.1.5 Finn

For the computation of the Cyclic Resistance Ratio **CRR**, Finn proposed a relationship in which the independent variables are the magnitude **M** and the corrected number of blows from the standard penetration test SPT $\mathbf{N_{1,60}}$:

$$CRR = \frac{N_{1.60}}{12.9\,M - 15.7}$$

Fs is obtained from:

$$F_S = \frac{CRR}{CSR}$$

CSR is calculated using the expression developed by **Seed and Idriss** (1971-1982) where the reduction factor r_d is determined using the empirical formula proposed by **Iwasaki et al.** (1978)

4.1.6 Cortè

For the computation of **CRR** , **Cortè** proposes two relationships function of the parameter $\mathbf{D_{50}}$:

- for soils with 0,04 mm ≤ D_{50} ≤ 0,6:

$$CRR = \left\{ \left[\frac{N_m}{\sigma'_{v0} + 70} \right]^{0.5} - 0.258 \log_{10}\left(\frac{D_{50}}{0.35} \right) \right\}$$

- for soils with 0,6 mm ≤ D_{50} ≤ 1,5:

$$CRR = \left\{ \left[\frac{N_m}{\sigma'_{v0} + 70} \right]^{0.5} - 0.0567 \right\}$$

The coefficient **A** takes values that vary between 0.50 and 0.66, depending on the magnitude of the earthquake and on the **number of equivalent cycles** that vary at their turn between 5 and 20.
CSR is calculated using the expression developed by **Seed and Idriss** (1971-1982) where the reduction factor $\mathbf{r_d}$ is determined using the empirical formula proposed by **Iwasaki et al.** (1978)

4.1.7 Robertson and Wride

The method of **Robertson e Wride** uses the soil behavior type index Ic that is calculated using the following formula:

$$I_c = \left[(3.47 - \log_{10} Q)^2 + (\log_{10} R_f + 1.22)^2 \right]^{0.5}$$

$$Q = \frac{q_c - \sigma_{v0}}{Pa} \left(\frac{Pa}{\sigma'_{v0}} \right)^n$$

$$R_f = \frac{f_s}{q_c - \sigma_{v0}} 100$$

where:

q_c measured point resistance

Pa reference stress (1 atmosphere) in same measurement units as σ'_{v0}.

f_s sleeve friction

n exponent depending on the soil type

Initially is assumed n = 1, as for a clayey ground and is proceed to the calculation of $\mathbf{I_c}$ with the formula above.

If I_C > 2.6 the soil is probably clayey type and the analysis stops. The soil is not considered at risk of liquefaction.
If I_C ≤ 2.6, means that the hypothesis assumed is incorrect, the soil has a granular nature, **Q** will be recalculated using the above relationship and using as an exponent n = 0.5.
If still I_C ≤ 2.6, means that the hypothesis is correct and the soil is probably not plastic and granular.
If instead I_C > 2.6, means that the hypothesis is wrong again and the soil is probably muddy. **Q** must be recalculated again putting n = 0.75.
Having calculated $\mathbf{I_c}$, we proceed with the correction of the cone resistance qc using the following expression:

$$q_{c1N} = \frac{q_c}{Pa} \left(\frac{Pa}{\sigma'_{v0}} \right)^n$$

Where the stress exponent n is the same used in the computation of $\mathbf{I_c}$.

The correction to the cone resistance due to the content of fine material is determined by:

$$(q_{c1N})_{cs} = K_c \cdot q_{c1N}$$

$$K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63\, I_c^2 + 33.75\, I_c - 17.88$$

The liquefaction resistance for a magnitude equal to 7,5 ($\mathbf{CRR_{7,5}}$) is calculated as:

- if $(qc1N)_{cs} < 50$:

$$CRR = 0.833\left[\frac{(q_{c1N})_{cs}}{1000}\right] + 0.05$$

- if $50 \leq (qc1N)_{cs} < 160$:

$$CRR = 93\left[\frac{(q_{c1N})_{cs}}{1000}\right] + 0.08$$

CSR is calculated using the formula mentioned in the Introduction chapter of this Guide, for different magnitude must be inserted the corrective factor **MSF** (Magnitude Scaling Factor) as recommended by **NCEER** (see Table 1 - Seed Idriss method).

To determine the values of the reduction factor **rd** are used the formulas recommended by an expert group of **NCEER** (*National Center for Earthquake Engineering Research*):

if z < 9,15 m:

$$r_d = 1.0 - 0.00765 \cdot z$$

if 9,15 ≤ z < 23 m:

$$r_d = 1.174 - 0.00267 \cdot z$$

where z is the depth in meters.

4.1.8 Robertson and Wride modified

In **Robertson e Wride**, the correction to the cone resistance due to the content of fine material is determined by the following procedure:

$$(q_{c1N})_{cs} = q_{c1N} + \Delta q_{c1N}$$
$$\Delta q_{c1N} = \frac{K_c}{1 - K_c} q_{c1N}$$

where K_c depends on the fine content, FC (%):

$$k_c = 0 \qquad \text{per } FC \le 5$$
$$k_c = 0.0267(FC - 5) \qquad \text{per } 5 < FC \le 35$$
$$k_c = 0.8 \qquad per\ FC > 35$$

FC (%) is calculated using the following formula:

$$FC(\%) = 1.75(I_c)^{3.25} - 3.7$$

The liquefaction resistance for a magnitude equal to 7,5 ($\mathbf{CRR_{7,5}}$) is calculated as:

if $(q_{c1N})_{cs} < 50$

$$CRR = 0.833\left[\frac{(q_{c1N})_{cs}}{1000}\right] + 0.05$$

if $50 \le (q_{c1N})_{cs} < 160$

$$CRR = 93\left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.08$$

CSR is calculated using the formula mentioned in the Introduction chapter of this Guide, for different magnitude must be inserted the

corrective factor **MSF** *(Magnitude Scaling Factor)* as recommended by **NCEER** (see Table 1 - Seed Idriss method).
To determine the values of the reduction factor **rd** are used the formulas recommended by an expert group of **NCEE** (*National Center for Earthquake Engineering Research*):

if z < 9,15 m:

$$r_d = 1.0 - 0.00765 \cdot z$$

if 9,15 ≤ z < 23 m:

$$r_d = 1.174 - 0.00267 \cdot z$$

where z is the depth in meters

4.1.9 Idriss and Boulanger (2008)

In the method proposed by **Idriss e Boulanger** the soil behavior type index IC is calculated using the following formulas:

$$I_c = \left[(3.47 - log_{10} Q)^2 + (log_{10} R_f + 1.22)^2 \right]^{0.5}$$

$$Q = \frac{q_c - \sigma_{v0}}{Pa} \left(\frac{Pa}{\sigma'_{v0}} \right)^n$$

$$R_f = \frac{f_s}{q_c - \sigma_{v0}} 100$$

where:
q_c - measured point resistance
Pa - reference stress (1 atmosphere) in same measurement units as σ'_{v0}.
f_s - sleeve friction
n exponent depending on the soil type

where **n** is determined iteratively by the following relation:

$$n = 1.338 - 0.249\, q_{c1N}{}^{0.264}$$

The correction to the cone resistance due to the content of fine material is determined by the following procedure:

$$(q_{c1N})_{cs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(5{,}4 + \frac{q_{c1N}}{16}\right) \cdot \exp\left[1{,}63 + \frac{9{,}7}{FC + 0{,}01} - \left(\frac{15{,}7}{FC + 0{,}01}\right)^2\right]$$

Where the fine content FC(%) is calculated with the following expression:

$$FS(\%) = 2.8 \cdot (I_c)^{2.60}$$

The liquefaction resistance for a magnitude equal to 7,5 (**CRR$_{7,5}$**) is calculated from:

$$CRR = exp\left[\frac{(q_{c1N})_{cs}}{540} + \left(\frac{(q_{c1N})_{cs}}{67}\right)^2 - \left(\frac{(q_{c1N})_{cs}}{80}\right)^3 + \left(\frac{(q_{c1N})_{cs}}{114}\right)^4 - 3\right]$$

For zw > z, with z_w groundwater table depth, and for (qc1N)cs ≤ 160 the soil is not liquefiable (NL).

CSR is calculated using the formula mentioned in the Introduction chapter of this Guide, for different magnitude must be inserted the corrective factor **MSF** (Magnitude Scaling Factor) as recommended by **NCEER** (see Table 1 - Seed Idriss method).

To determine the values of the reduction factor **rd** are used the following formulas:

$$r_d = \exp[\alpha(z) + \beta(z) \cdot M]$$

$$\alpha = -1{,}1012 - 1{,}126 \cdot sen\left[\frac{z}{11{,}73} + 5{,}133\right]$$

$$\beta = 0{,}106 + 0{,}118 \cdot sen\left[\frac{z}{11{,}28} + 5{,}142\right]$$

$$MSF = 6{,}9 \cdot \exp\left(-\frac{M}{4}\right) - 0{,}058 \le 1{,}8$$

The liquefaction factor of safety FS is determined by:

$$FS = \frac{CRR_{7,5}}{CSR} \cdot MSF \cdot K_\sigma$$

To determine the magnitude scaling factor **MSF**, the formula of **Idriss & Boulanger** uses the expression:

$$MSF = 6{,}9 \cdot \exp\left(-\frac{M}{4}\right) - 0{,}058 \leq 1{,}8$$

The correction factor of the confining pressure K_σ is given by:

$$K_\sigma = 1 - C_\sigma \cdot \ln\left(\frac{\sigma'_{v0}}{p_a}\right) \leq 1$$

$$C_\sigma = \frac{1}{37{,}3 - 8{,}27 \cdot (q_{c1N})^{0{,}264}} \leq 0{,}3$$

4.1.10 Andrus and Stokoe

The method of **Andrus e Stokoe** is based on measurements from seismic refraction tests, (Vs).
The velocity of the shear waves is corrected by the overpressure, using the equation (Robertson et al., 1992):

$$V_{s1} = V_S \left(\frac{100}{\sigma'_{v0}}\right)^{0.25}$$

where
V_{S1} shear waves velocity corrected by the overpressure

V_S shear waves velocity measured in situ

Pa atmospheric pressure (about 100 kPa)

σ'_{vo} effective initial pressure in the same units of measure as Pa

For the calculation of the resistance to liquefaction, Andrus and Stokoe have proposed the following relationship:

$$CRR = 0.03 \cdot \left(\frac{V_{S1}}{100}\right)^2 + 0.9 \cdot \left[\frac{1}{(V_{S1})_{cs} - V_{S1}} - \frac{1}{(V_{S1})cs}\right]$$

Where the presence of fine content FC (%) intervenes in the calculation model by the following specifications:

$$
\begin{array}{ll}
(V_{S1})_{cs} = 220 & \text{per } FC \le 5\% \\
220 < (V_{S1})_{cs} \le 200 & \text{per } 5\% < FC \le 35\% \\
(V_{S1})_{cs} = 200 & \text{per } FC > 35\%
\end{array}
$$

CSR is calculated using the formula mentioned in the Introduction chapter of this Guide, for different magnitude must be inserted the corrective factor **MSF** (Magnitude Scaling Factor) as recommended by **NCEER** (see Table 1 - Seed Idriss method).

4.1.11 EC8

The indications of European legislation are found in paragraph 4.1.3 with further directions that can be found in Appendix B of Part 5 of EC8.
According to this legislation can be excluded the risk of liquefaction for saturated sandy soils that are found at depths of 15 m or when ag < 0.15 and, at the same time, the soil meets at least one of the following conditions:

- clay content greater than 20%, with plasticity index>10
- silt content greater than 10% and $N_{1,60}$ > 20
- fine fraction negligible and $N_{1,60}$ > 25

Generally the method is valid if $N_{1,60}$ < 30. For $N_{1,60}$ > 30, the soils are classified not liquefiable (clean dense granular soils).

When none of the above conditions is met, *the susceptibility to liquefaction shall be verified as minimum by generally accepted methods of the geotechnical engineering, based on correlations between in situ measurements and the critical values of cyclic shear stress that caused liquefaction during past earthquakes.*

The cyclic stress ratio **CSR** is calculated using the simplified formula:

$$CSR = 0.65 \frac{a_g}{g} \cdot S \frac{\sigma_{v0}}{\sigma'_{v0}} \frac{r_d}{MSF}$$

where **S** is the stratigraphic profile coefficient, defined as:

Table 5- *Stratigraphic profile coefficients*

Soil type	*Spectra of Type 1 S (M > 5,5)*	*Spectra of Type 2 S (M ≤ 5,5)*
A	*1,00*	*1,00*
B	*1,20*	*1,35*
C	*1,15*	*1,50*
D	*1,35*	*1,80*
E	*1,40*	*1,60*

The magnitude scaling factor **MSF** suggested by the legislation is **Ambraseys** (Table 1-method Seed Idriss).

If there is any data from the **SPT** tests, liquefaction resistance is calculated using **Blake** (1997) equation:

$$CRR = \frac{0,04844 - 0,004721\,(N_{1,60})_{cs} + 0,0006136\,\left[(N_{1,60})_{cs}\right]^2 - 0,00001673\,\left[(N_{1,60})_{cs}\right]^3}{1 - 0,1248\,(N_{1,60})_{cs} + 0,009578\,\left[(N_{1,60})_{cs}\right]^2 - 0,0003285\,\left[(N_{1,60})_{cs}\right]^3 + 0,00000371\,4\,\left[(N_{1,60})_{cs}\right]^4}$$

where $(N_{1,60})_{cs}$ is calculated using the method proposed by **Youd and Idriss (1997)** and recommended by **NCEER**:

$$(N_{1.60})_{cs} = \alpha + \beta\, N_{1.60}$$

where $\mathbf{N}_{1,60}$ is the normalization of the measured values of the index $\mathbf{N_m}$ (reduced by 25% for depth < 3 m) in the SPT test compared to an effective pressure of 100 KPa and a value of the ratio between the impact energy and the theoretical energy of free fall (flight) equal to 60%, ie:

$$N_{1,60} = C_N\, C_E\, N_m$$

$$C_N = \left(\frac{100}{\sigma'_{vo}}\right)^{0,5}$$

$$C_E = \frac{ER}{60}$$

$$C_E = \frac{ER}{60}$$

where **ER** is equal to (ratio of the measured energy compared to the theoretical value) x 100 and depends on the type of equipment used (Table 6).

Table 6- *Drilling system efficiency*

Equipment	C_E
Safety Hammer	0,7÷1,2
Donut Hammer (USA)	0,5÷1,0
Donut Hammer (Japan)	1,1÷1,4
Automatic-Trip Hammer (Type Donut or Safety)	0,8÷1,4

The parameters α and β, instead, depend on the fine fraction FC:

$\alpha = 0$ for $FC \leq 5\%$

$\alpha = \exp[1,76 - (190 / FC^2)]$ for $5\% < FC \leq 35\%$

$\alpha = 5$ for $FC > 35\%$

$\beta = 1,0$ for $FC \leq 5\%$

$\beta = [0,99 + (FC^{1,5} / 1000)]$ for $5\% < FC \leq 35\%$

$\beta = 1,2$ for $FC > 35\%$

In the following cases, the soil **is not susceptible to liquefaction**, the program does not provide the safety factor but (--):

1. $a_{max} < 0.15$, Clay fraction > 20 and IP = > 10
2. Number of the CRR report less than zero
3. $a_{max} < 0.15$, Clay fraction > 20
4. $a_{max} < 0.15$, $N_{1,60} > 25$

If data from a static penetration test (**CPT**) are available, the measured tip resistance (qc) values must be normalized compared to an confinement effective pressure equal to 100 KPa and must be calculated using the following relationship:

$$q_{c1N} = \frac{q_c}{Pa}\left(\frac{Pa}{\sigma'_{vo}}\right)^n$$

As proposed by **EC8**, when data from a **CPT** test is available, can be used the following equation to derive the value of (N1,60)cs:

$$\frac{(q_{c1N})_{cs}}{(N_{1,60})_{cs}} = 5$$

The value of the liquefaction resistance is determined by the relationship of **Blake** (1997). When instead is available data from refraction seismic tests, the normalized propagation velocity is calculated using the relationship proposed by of **Robertson et al.** (1992):

$$V_{S1} = V_S \left(\frac{Pa}{\sigma'_{vo}} \right)^{0,25}$$

For the liquefaction resistance is used the formula of **Andrus and Stokoe** :

4.1.12 Liquefaction potential index LPI

The liquefaction potential index LPI is a measure of the liquefaction effects based on the width and depth of the liquefiable areas and on historical cases of liquefaction.
The methods implemented in ***LIQUITER*** for the calculation of the liquefaction potential LPI are: Iwasaki et al. (1982) and Sonmez (2003).
The calculation of the liquefaction potential index is defined by:

$$IPL = \int_0^{z_{crit}} F(z) \cdot w(z) \cdot dz$$

Iwasaki

F(z) is function of the safety factor that for:

$$F(z) = 0 \quad \text{if} \quad \mathrm{FSL} > 1$$
$$F(z) = 1 - FSL \quad \text{if} \quad \mathrm{FSL} < 1$$

Sonmez

$$F(z) = 0 \quad \text{if} \quad \mathrm{FSL} \geq 1.2$$
$$F(z) = 2 \cdot 10^6 \cdot \exp(-18.427 \cdot FSL) \quad \text{if} \quad 1.2 > \mathrm{FSL} > 0.95$$
$$F(z) = 1 - FSL \quad \text{if} \quad \mathrm{FSL} \leq 0.95$$

Is indicated with z_{crit} the maximum depth to which liquefied layers produce effects in surface:

$$\mathrm{Se} \quad z_{crit} = 20\,\mathrm{m} \quad w(z) = 10 - 0.5 \cdot z$$
$$\mathrm{Se} \quad z_{crit} = 10\,\mathrm{m} \quad w(z) = 20 - 2 \cdot z$$

Traditionally, the critical depth is assumed to be 20 m, but recently Ozocak and Sert (2010), based on experimental evidence successive to earthquakes in Adapazari (Turkey) in 1999 and based on the limit curves for manifestations of surface liquefaction in Ishihara (1985), have proposed to take for earthquakes of "usual" magnitude the critical depth of 10 m.

The classes of liquefaction potential, according to the proposal Sonmez (2003), are the following:

LPI	*Liquefaction potential*
0	*Non liquefiable*
0<LPI≤2	*Low*
2<LPI≤5	*Moderate*
5<LPI≤15	*High*
LPI>15	*Very high*

4.2 Limit State of Liquefaction C. Hsein Juang 2006

The limit state for liquefaction triggering is obtained using a neural network-based searching technique developed by **Juang et al. 2000b** . The technique involves the training of supervised feed-forward neural networks with the *"full"* database of case histories and its subsets or samples. The successfully trained neural network that generates the most accurate *input–output* relationship is adopted in the subsequent step for searching *"data points"* on the unknown boundary surface. Regression analyses of the searched data points, with some engineering judgment, yields the following empirical equation for liquefaction resistance:

$$CRR = \exp(-2.8781 + 0.000309 \cdot (qc1N,m)^{1.81}) \quad (1)$$

where:

qc1N,m=stress-normalized cone tip resistance qc1N adjusted for the effect of *"fines"* on liquefaction thus, qc1N,m=K·qc1N.

The stress-normalized cone tip resistance qc1N used herein follows the definition by **Idriss and Boulanger 2004**, although the difference between this definition and that by **Robertson and Wride 1998** is rather small for the cases examined.

K is part of the regression model and expressed as:

$$K = 1 \text{ for } Ic < 1.64 \quad (2)$$

$=1 + 80.06\cdot(Ic - 1.64)\cdot(qc1N)^{-1.2194}$ for $1.64 < Ic < 2.38$

$=1 + 59.24\cdot(qc1N)^{-1.2194}$ for $Ic > 2.38$

Both Ic and qc1N in **Eqs. 2** are dimensionless. The term Ic in **Eqs. 2** is a variant of the soil behavior type index defined by **Lunne et al. 1997** and **Robertson and Wride 1998**, and updated in **Zhang et al. 2002**.

Although Ic was initially developed for soil classification, use of Ic to *"gage"* the effect of *"fines"* on liquefaction resistance is well accepted **Robertson and Wride 1998; Youd et al. 2001;**

As with any simplified methods that follow the general framework by **Seed and Idriss 1971**, CRR is defined as the critical CSR that causes liquefaction for a given soil. Thus, it is essential
that the CRR equation has to be used along with the reference CSR equation. To use **Eq. 1** for determination of CRR, the following cyclic stress ratio model must be used:

$$CSR_{7.5\sigma} = 0.65\cdot(\sigma_v/\sigma'_v)\cdot(a_{max}/g)\cdot r_d\cdot(1/MSF)\cdot(1/K\sigma) \quad (3)$$

where:

g= acceleration of gravity, which is the unit for amax;
r_d= depth-dependent shear stress reduction factor dimensionless;
MSF= magnitude scaling factor dimensionless;
Kσ= overburden correction factor dimensionless for CSR.

In Eq. 3, $CSR_{7.5\sigma}$, is the CSR defined by **Seed and Idriss 1971** adjusted to the conditions of Mw moment magnitude=7.5 and σ=100 kPa. Such adjustment makes it easier to process case histories from different earthquakes and with soils of concern at different overburden pressures **Juang et al. 2003.** It should be noted that in this paper, the terms rd, MSF, and Kσ are calculated with the formulae recommended by **Idriss and Boulanger 2004** ;

The method C. **Hsein Juang** was implemented in software LIQUITER

More details

4.3 Interventions

4.3.1 Gravel drains

Columns of gravel are inserted in the liquefiable layer and are usually installed in quincunx (Figure 5a), as this is the cheapest disposition available. In practice, however, are also arranged in a square mesh (Figure 5b).

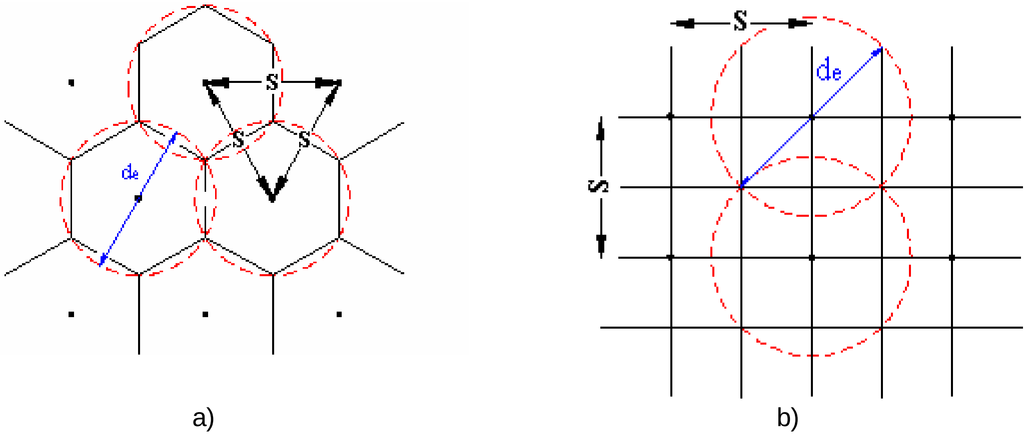


Figure 5 *– Disposition of drains: a) Triangular disposition (quincunx); b) Square mesh disposition. S indicates the spacing between the drains, while d_e is the equivalent diameter of drained soil cylinder*

S indicates the spacing between the drains, while $\mathbf{d_e}$ is the equivalent diameter of drained soil cylinder.

In any case, the problem to be solved can be reduced to that of a soil equivalent (Figure 6), with the waterproof outer lateral surface and a central drain.

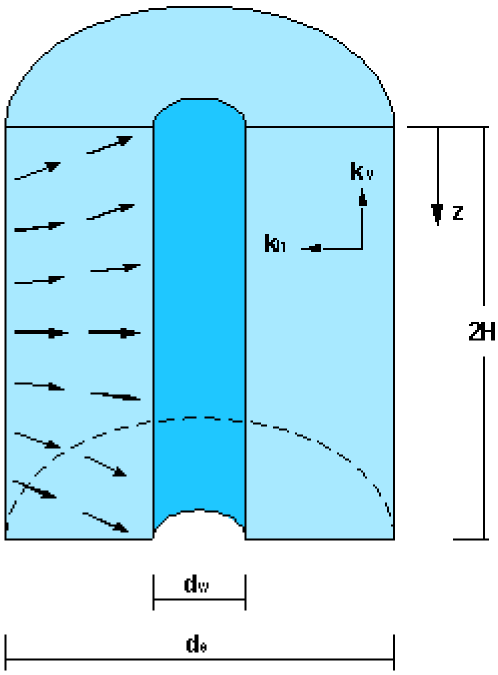


Figura 6 *– Scheme of drained soil equivalent cylinder*

d_w = drain diameter

d_e = drained soil equivalent cylinder diameter

k_h = permeability coefficient in horizontal direction of the soil in undisturbed conditions

k_v = permeability coefficient in vertical direction of the soil in undisturbed conditions

2H = height from drain

z = relative depth

The equivalent diameter of the soil cylinder that drains de is equal to 1.05 times the spacing **S** of the drains if they are arranged in quincunx and equal to 1.13 **S** in case they have a square mesh disposition.
For drains in square mesh disposition it is possible to evaluate the spacing necessary to bring the void ratio from a value **eo** to a value e in an approximated way with the following expression:

$$S = \left[\frac{1 - e_o}{e_o - e} \right]^{0,5} d_w$$

Barron (1948) was the first to develop a systematic and complete approach of the problem; in it are taken as valid the assumptions of Terzaghi's one-dimensional theory.

The average degree of consolidation $\mathbf{U_h}$ is calculated using the following expression:

$$U_h = 1 - \exp\left(-\frac{8T_h}{F}\right)$$

where $\mathbf{T_h}$ and **F** are equal to:

$$T_h = \frac{k_h}{\gamma_w}\frac{t}{m_v\, d_e^2}$$

$$F = \frac{n^2}{n^2-1}\ln(n) - \frac{3n^2-1}{4n^2}$$

where:

$\mathbf{m_v}$ = coefficient of volumetric compressibility

n = ratio between diameter de and diameter dw

t = 0,055 exp(0,861M) duration of the design seismic event

M = magnitude of the design seismic event

The magnitude of the design earthquake is calculated using the empirical relationship of **Berardi et al.**

$$\log R = 0{,}77\,M - 3{,}6$$

where **R** is the epicentral distance in km of the design earthquake.

This report has the meaning of minimum magnitude required to produce liquefaction of recent surface saturated sand deposits and allows to work in favor of safety.

4.3.2 Heavy tamping

The method *heavy tamping* consists in producing an increase in the relative density of liquefiable soils by free fall from heights of up to 30-40 meters of large concrete or steel blocks weighing up to tens of tons, causing compression waves due to sudden release of energy, which generate an instantaneous increase of pore pressure, reducing the shear strength in the soil by inducing a series of subsequent liquefaction. When the excess pore pressure dissipates, the particles reach new, more stable, configurations.

The procedure normally requires 2-3 shots per m2. At the end is advisable to run a check, for example, penetration tests, in order to establish that the soil has actually achieved an increase of the relative density. The tests will be performed up to a depth of densification influence, depending on the weight of the mass W and the fall height H and is evaluated by the empirical expression:

$$D = (0,65 - 0,80)WH$$

where **W** is measured in tons **D** and **H** in meters.

The success of this method on natural soils is not always guaranteed, especially if there is a percentage of fine content greater than 10%; instead were obtained excellent results in densification of landfills.

The method heavy tamping is simple and quick at acceptable costs, from which it also derives a good uniformity of treatment. However, it can't be used in the vicinity of existing structures given that the vibrations produced induce harmful effects on the structures.

4.4 Lateral Spreading

4.4.1 Lateral Spreading

The loss of resistance in granular saturated soils due to the phenomenon of liquefaction is the cause of large horizontal deformations of the ground.
Lateral spreading consists of a lateral movement of large ground blocks above the liquefied level, which occurs when are shared flat or gently sloped soils (0°-3°) of alluvial materials.
The top layer will fracture into blocks and the material that constitutes the lower layer (liquefied) goes to fill the fractures.
The fractured soil moves laterally toward the free surface with even metric displacements.

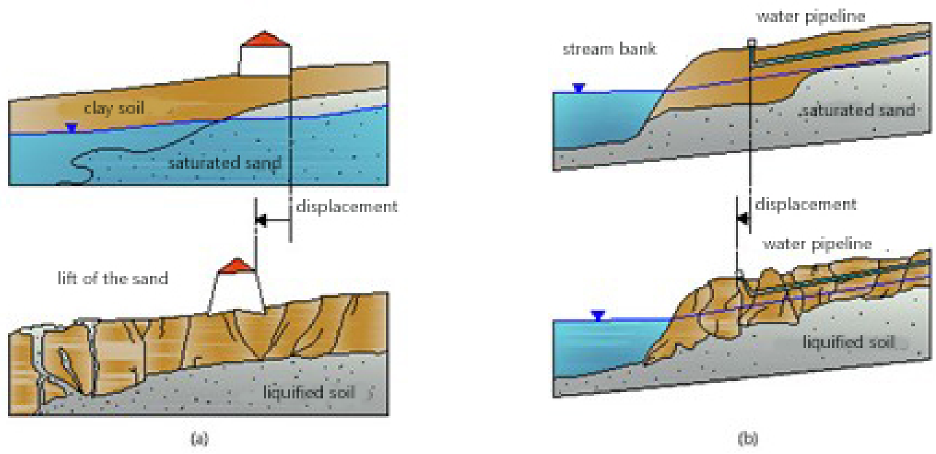


Figure 7- *(a)Liquefaction of soil with lateral movement, (b) movement with advancement front*

Bartlett and Yound (1995) have obtained two independent models for the estimation of Lateral spreading (valid for (N1)60 less than 15 or for a distance from the source under 30 Km):

- Model with flat surface for areas near the banks

$$log\Delta h = -16.3658 + 1.178 \cdot M - 0.9275 \cdot logr - 0.0133 \cdot r + 0.6572 \cdot logW + 0.3483 \cdot logH_{15} + 4.5270 \cdot \log(100 - F_{15}) - 0.9224 \cdot D_{50_{15}}$$

- Model with inclined surface for areas with slightly inclined soils

$$log\Delta h = -15.7870 + 1.1782 \cdot M - 0.9275 \cdot logr - 0.0133 \cdot r + 0.4293 \cdot logs + 0.3483 \cdot logH_{15} + 4.5270 \cdot \log(100 - F_{15}) - 0.9224 \cdot D_{50_{15}}$$

where:

Δh = value in meters of the soil lateral displacement

H$_{15}$ = cumulative thickness of the saturated layers with the corrected number of blows, (N1)60, less than 15, (in meters)

(**D**$_{50}$)$_{15}$ = average size D50 of granules included in H15, in millimeters

F$_{15}$ = average of fine content (fraction of sediment passing through the sieve n° 200) of the layers contained in H15

M = magnitude of earthquake

R = horizontal distance from the source of seismic energy

s = slope of the soil

W = ratio between the height of the free surface (H) and the distance between the foot of the free surface and the site considered (L)

By analyzing historical cases, the authors **Bartlett and Yound** (1992) have identified the variation range of the variables contained in the above equations, necessary for the occurrence of the *Lateral spreading* phenomenon (see Table 6).

Table 6- *Range of values of the variables in the equations of Bartlett and Yound (1995)*

Factor	Values
Magnitude	$6.0 < M < 8.0$
Ratio height/distance	$1.0\% < W < 20\%$
Ground slope	$1.0\% < s < 6\%$
Thickness of the loose layer	$0.3 < H_{15} < 6$ m
Fine content	$0\% < F_{15} < 50\%$
Average grain size	0.1 mm $< (D_{50})_{15} < 1$ mm
Depth of the section bottom	< 15 m

Choosing the command ***"Lateral spreading"***, entering the required input data and clicking the ***"Computation"*** button the software calculates the lateral displacement using the computation models described above.
The button ***"Report"*** offers the possibility to print theoretical content, a summary of input data and the results.

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6 Utility

6.1 Conversion Tables

Converting slope inclination into degrees and vice versa

Inclination (%)	Angle (°)	Inclination (%)	Angle (°)
1	0.5729	26	14.5742
2	1.1458	27	15.1096
3	1.7184	28	15.6422
4	2.2906	29	16.1722
5	2.8624	30	16.6992
6	3.4336	31	17.2234
7	4.0042	32	17.7447
8	4.5739	33	18.2629
9	5.1428	34	18.7780
10	5.7106	35	19.2900
11	6.2773	36	19.7989
12	6.8428	37	20.3045
13	7.4069	38	20.8068
14	7.9696	39	21.3058
15	8.5308	40	21.8014
16	9.0903	41	22.2936
17	9.6480	42	22.7824
18	10.2040	43	23.2677
19	10.7580	44	23.7495
20	11.3099	45	24.2277
21	11.8598	46	24.7024
22	12.4074	47	25.1735
23	12.9528	48	25.6410
24	13.4957	49	26.1049
25	14.0362	50	26.5651

Forces conversion

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

From	To	Operation	Factor
Tone	kN	Multiply by	9.8

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

Pressures conversion

From	To	Operation	Factor
Tons/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

1 Pascal (Pa) = 1 Newton/mq ; 1 kPa = 1000 Pa

6.2 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/cm^2	1.20	2.40
Calyey soil with 2< qu< 4 Kg/cm^2	2.20	4.80
Calyey soil with qu> 2 Kg/cm^2	>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

Soil	Maximum value of E	Minimum value of E
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 elastic 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
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

Rock	Minimum value	Maximum value
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 j, 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

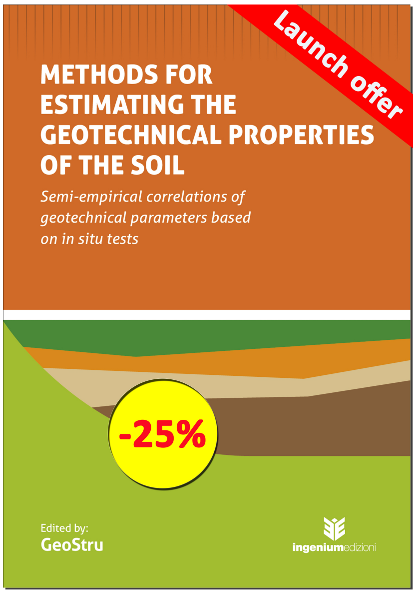
7 Recommended books

Geotechnical, engineering, and geology books

Portal books: explore the library

• Methods for estimating the geotechnical properties of the soil

Methods for estimating the geotechnical properties of the soil: semi-empirical correlations of geotechnical parameters based on in-situ soil tests.
This text is designed for all professionals who operate in the geotechnical subsurface investigation. The purpose of this text is to provide an easy reference tool relatively to the means available today.
Theoretical insights have been avoided, for which please refer to the bibliography attached, except in cases where these were considered essential for the understanding of the formulation. The reason for this is obvious: make the text as easy to read as possible.
After a brief introduction about volumetric and density relationships with the most common definitions used for soils, in the following chapters we briefly described some of the most widespread in situ geotechnical testing and correlations to derive empirically geotechnical parameters and a number of useful formulations available today in the field of Geology.
The text concludes with the inclusion of formulas used in Technical Geology, considered of daily use to those working in the sector.
The topics are intended to provide a basic understanding of the in situ geotechnical testing and evaluation of geotechnical parameters necessary to define the geotechnical model.



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.

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8.1 Geoapp Section

General and Engineering, Geotechnics and Geology

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

- Sismogenetic zone
- Soil classification SMC
- Seismic parameters
- NSPT
- Slope stability
- Landslide trigger
- Critical heigh (maximum depth that can be excavated without failure)
- Bearing capacity
- Lithostatic tensions
- Foundation piles, horizontal reaction coefficient
- Liquefaction (Boulanger 2014)

9 Contact

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