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Parametric analysis of reinforced earth slopes varying seismic acceleration and soil foundation properties

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1. INTRODUCTION

This work is aimed at testing some ultimate limit states for reinforced earth using the Technical Standards for Construction (NTC) from D.M. 14/01/2008 in relation to the geotechnical aspects.

We will try to give a critical judgment on safety factors that are going to vary for the condition of the soil and the seismic acceleration, using a comparison between:

• D.M. 14/01/2008 (DA1.C1 ; DA1.C2)
In Chapter 2, we introduce the most significant features of the legislation with particular focus to:
• The introduction of the semi-probabilistic limit state method for the verification of safety with the application of various design approaches and their partial safety factors;
• The determination of the local seismic response for the evaluation of the seismic project related to the introduction of specific rules for design and verification of works subject to such actions.

In Chapter 3, we will introduce the technique of soil reinforcement by reference to the theoretical principles and practical aspects.

Chapter 4 will present a search for the optimal conditions for the realization of reinforced soil work. In these cases the water will be absent while the concentrated load and height are set. Through a parametric study, will be search for each design approach required by the NTC 2008, the minimum base width with the ratio B / H, which allows to fulfill the ultimate limit states considered.
The values obtained are compared in graphs and make a few personal observations on various safety factors to take into consideration for the optimization of the slope.
The parametric study will be conducted on works that are located at different sites in seismic intensity and similarly will be given a critical judgment about the results.

In Chapter 5 you will see the various methods used for such research and optimization will be done with a specific report of the work.

In conclusions, Chapter 6, we will expose the limitations in the treatment carried out and you will locate a logical thread with the orientation of the ministerial circular based on the results obtained in this paper.

2. Technical Standards for Construction M.D. 14/01/2008

2.1 INTRODUCTION TO THE LIMIT STATES

The Technical Standards for Construction (NTC), in force with the DM 14/01/2008, born on the settings of the Eurocodes, deal in a uniform way both the geotechnical and structural design.
This represents a significant change from the old national legislation, as these arguments were always remained divided both in a physical manner (different regulations) that especially in the general theory.
The works and the various structural types (including those geotechnical) must have the following requirements [2.1 DM 2008]:

- safety against ultimate limit state (ULS): ability to avoid crashes, loss of balance and severe disruptions, total or partial, that could compromise the safety of individuals or result in the loss of property, or causing serious environmental and social damage or put out of service work;

- security against serviceability limit states (SLS): ability to ensure the expected performance for the operating conditions;

- robustness against actions exceptional ability to avoid damage.

The NTC then recite [2.1 DM 2008]: "For the assessment of the safety of the works of construction should be adopted probabilistic criteria scientifically proven. The following are the normative criteria of the semi-probabilistic limit state method based on the use of partial safety factors, applicable in most cases, this method is called first level. "

In this way, with the D.M. 14.01.2008 (in analogy with all the Eurocodes), it confirms the concept that it is possible to design risk-free but only if the risk of failure is below a certain value. Technically, fixed the probability of failure, this corresponds to identify for each state limit, the partial safety factors to be assigned to the individual quantities that affect that particular state limit.

The meaning of the partial coefficients, as statistical quantities, is therefore to quantify the weight with which each parameter helps to make sure that the probability of failure of the work does not exceed the predetermined design value.

While in structural variability of physical and mechanical properties of materials, the geometry of the structure, methods and models of computation elements are quite certain, this is not the same for the geotechnical engineering.

In the project geotechnical uncertainties on independent random variables, models and methods of calculation are much larger and less the result of past experience, a process, with all the simplifications that correct or not you are obliged to do, it can be seen that:

- analysis and understanding of physical reality in which the work is positioned;
- identification of the main factors that influence the behavior;
- schematic of the possible design problems;
- formulation of a suitable mathematical model;
- implementation with choice of design parameters;
- critical evaluation of the results.

In other words, we can say that the reliability of the result is conditioned by the weakest link in the chain.

Perhaps for this reason, whereas the geotechnical construction is not characterized by a unique and shared, the NTC for it suggests two possible approaches to the design, thereby giving weight to the sensitivity, responsibility and experience of the designer. This is also reflected in many other parts of the legislation, for example:

[6.2.2. DM 2008] "For the characteristic value of a geotechnical parameter to be considered a reasoned estimate of the value of the parameter and cautionary limit state considered."

[6.2.2. DM 2008] "It is the responsibility of the designer's defined benefit plan of the investigation, characterization and geotechnical modeling."

[6.3.4. DM 2008] "The degree of safety acceptable to the designer must be justified on the basis of the level of knowledge attained, the reliability of available data and the computational model adopted in relation to the complexity of geological and geotechnical engineering, as well as on the basis of the consequences of a 'eventual landslide. "

2.2 THE LIMIT STATES IN THE SUPPORT WORKS

To support works are:

• WALLS: for which the support function is entrusted to the own weight of the wall and that of the soil directly acting on it;

• PARATIE: for which the support function is ensured mainly by the resistance of the volume of soil placed before the work and possible anchors and struts;

• MIXED STRUCTURES: exert support function also for the effect of treatments for improvement and for the presence of particular elements of reinforcement and connection.

We consider actions those due to the own weight of the soil and of the filling material, overloads, water, for any pre-stressed anchorages, to wave motion, to shocks and collisions, to temperature variations and to the ice.
It must be understood that the soil and water are permanent loads (structural) when used in the modeling, to contribute to the behavior of the work with their weight characteristics, strength and stiffness.
In assessing the overload on the back of a work support should be taken into account the possible presence of buildings, deposits of material, passing vehicles, lifting equipment.
The geometrical model of the work of support shall take account of possible changes in the level of the land upstream and downstream of the work from the nominal values.

To limit state means "the condition after which the work don’t meets the conditions for which it was designed." Limit states may be (ULS) or (SLS). Exceeding either the ultimate limit state is irreversible and is defined collapse. Overcoming a state operating limit may be of a reversible or irreversible.
As a first approximation we can say that the ultimate limit state verification always guarantees at break, while checking the serviceability limit state guarantees against excessive deformation.

2.2.1 CHECKING AT THE ULTIMATE LIMIT STATES

For each ultimate limit state must be the condition:

$$E_d \leq R_d$$

Where Ed is the design value of the action:

$$E_d = E\left[\gamma_F \cdot F_k ; \frac{X_k}{\gamma_M} ; a_d\right]$$

o dell'effetto dell'azione:

$$E_d = \gamma_E \cdot E\left[F_k ; \frac{X_k}{\gamma_M} ; a_d\right] \text{ with } \gamma_E = \gamma_F$$

while Rd is the design value of the resistance of the geotechnic system:

$$R_d = \frac{1}{\gamma_R} \cdot R\left[\gamma_F \cdot F_k ; \frac{X_k}{\gamma_M} ; a_d\right]$$

Effect of action and resistance are expressed in terms of design actions γ F ⊙ Fk, project parameters Xk/γ_M and geometry to project $a_{d.}$ The effect of the actions can also be directly assessed as Ed = $Ek\gamma_E$. In the formulation of the resistance Rd, explicitly mentioned a coefficient γ_R which operates directly on the strength of the system.
Regard to are pointed to the geometric characteristics of the design of the structure and of the subsoil (including also the level of the free surface of a possible aquifer); these dimensions may be different from those characteristics, to take account of uncertainties on geometry stratigraphy or the layer level:

$$a_d = a_k \pm \Delta a$$

In verifying the supporting structures are distinguished:

• limit state of equilibrium as a rigid body: EQU

• limit state resistance of the structure including the foundation: STR
• limit state resistance of the soil: GEO
• limit state of hydraulic type: HYD

For verification against the ultimate limit state of equilibrium as a rigid body (EQU) using partial factors γ F relating to the actions listed in column EQU Table 2.1.

In tests against the ultimate limit state structural (STR) and geotechnical (GEO) can be taken, alternatively, two different design approaches are described below.

2.2.1.1 Design approach 1 [DA1]

1 is in the approach employing two different combinations of groups of partial coefficients, respectively defined for the actions (A1 and A2), for the resistance of materials (M1 and M2) and, possibly, to the overall strength of the system (R1, R2 and R3).

• APPROACH 1 COMBINATION 1 [DA1 C1]

For actions it takes the coefficients γ F in column A1 of Table 2.1. This combination is generally more severe in relation to the structural dimensioning of the works in contact with the ground.

• APPROACH 1 COMBINATION 2 [DA1 C2]

For actions it takes the coefficients γ F in column A2 of Table 2.1. This combination is generally more severe in regard to the sizing geotechnical.

2.2.1.2 Design approach 2 [DA2]

Approach 2 is used a unique combination of groups of coefficients defined for the Shares (A1), for resistance of materials (M1) and for the global resistance (R3). This approach is not mentioned in the checks for the bulkheads.

The following are the summary tables can be used for verification ULS supporting structures, summarizing the partial factors for actions (A1 and A2), for geotechnical parameters (M1 and M2) and the resistance of the system (R1 , R2 and R3).

Table 2.1 – *Partial coefficients for actions or for actions effects.*

LOADS	EFFECT	Partial Coefficient γ_F (o γ_E)	EQU	**(A1)** STR	**(A2)** GEO
Permanent	Favorable	γ_{G1}	0,9	1,0	1,0
	Unfavorable		1,1	1,3	1,0
Permanent not structural	Favorable	γ_{G2}	0,0	0,0	0,0
	Unfavorable		1,5	1,5	1,3
Variable	Favorable	γ_{Qi}	0,0	0,0	0,0
	Unfavorable		1,5	1,5	1,3

Table 2.2. –*Partial factors for soil parameters*

PARAMETER	QUANTITY TO WHICH APPLY THE PARTIAL COEFFICIENT	Partial coefficient γ_M	**(M1)**	**(M2)**
Angle of shearing resistance	$\tan \varphi'_k$	$\gamma_{\varphi'}$	1,0	1,25
Effective cohesion	c'_k	$\gamma_{c'}$	1,0	1,25
Undrained shear strenght	c_{uk}	γ_{cu}	1,0	1,4
Weight density	γ	γ_γ	1,0	1,0

Table 2.3 - ***Partial factors for the ultimate limit state checks STR e GEO.***

CHECKS	Partial coefficient **(R1)**	Partial coefficient **(R2)**	Partial coefficient **(R3)**
Bearing capacity of the foundation	γ_R = 1,0	γ_R = 1,0	γ_R = 1,4
Sliding	γ_R = 1,0	γ_R = 1,0	γ_R = 1,1
Soil resistance downstream	γ_R = 1,0	γ_R = 1,0	γ_R = 1,4

Note:

With the approved version of Eurocode 7 in order to take account of the conflicting demands of the major European countries (especially England, France, Germany), each interested in safeguarding national planning habits, it has come to the conclusion to predict three possible approaches by design.

The third approach is the following: the partial safety factors are applied to the shares (or the effects of actions) and the parameters of the soil. As regards the measures, however, a distinction is made between those resulting from the geotechnical structures and those of origin, that is, those actions carried out by the natural terrain, from filling of land and groundwater. In the first case, in fact, all the unfavorable permanent actions are amplified, while in the second case the shares must be taken

with their characteristic value, ie not amplified. Regarding these unfavorable variable actions are amplified, even if with different partial coefficients, in both cases.
The EC7 then establishes that each country can choose which approach (or approaches) also adopt differentiating by type of geotechnical work.
The Italian national document in particular suggested for the design and verification only the first two approaches.

2.2.2 CHECKS TO THE LIMIT STATES

The works of support and in general all geotechnical systems must be checked against the limit state. To this end, the project should clarify the requirements relating to movements compatible and performance expectations for the work itself.
For each limit state must be the condition: Ed ≤ Cd.
where Ed is the design value of the shares and Cd is the prescribed limit value of the effect of actions. The latter must be established on the basis of the behavior of the structure in elevation.
The use of SLE represents a significant innovation in the geotechnical field, because with this choice requires that the design of geotechnical work is not only conditioned by security against possible limit states of collapse but also by the need to ensure the functionality and usability in time of the work itself.
It thus implies the evaluation of the possible expected displacements as a function of the deformations of the ground and a comparison between these and the limit value in this case.
At times, the boundary condition on the functionality is more restrictive than that resulting from the ultimate limit states.
With regard to the practice, the checks are carried out to SLU accordance with methods of limit equilibrium, using a pattern of behavior rigid-perfectly plastic that does not allow the estimation of the deformations and displacements. In this context, then, we can say that the method allowable stresses has never really been used, just by virtue of the study of land whose behavior is not linear elastic even at low levels of deformation.
In essence, the ultimate limit state checks are part of the tradition geotechnical, while checks at serviceability limit state were increasingly rare.
Only in relatively recent times with the development of elastoplastic constitutive models for soils, with the evolution of the techniques of in situ and laboratory testing, and with the broadcast of the finite element or finite differences is possible to estimate the field of stress and strain and their development over time.
Even today, however, for works of ordinary importance using limit equilibrium methods.
Consequently, the limit state checks for the year, in geotechnical engineering, are often carried out with non-analytical and empirical methods.

2.2.3 CHECK TO THE HYDRAULIC LIMIT STATES

The works of support must be verified against the possible limit states for lifting or siphoning. For stability for lifting must be that the design value of the action instabilizzante Vinst, d, combination of permanent actions (Ginst, d) and variable (Qinst, d) is not greater than the combination of the design values of actions stabilizers (GSTB, d) and of the resistors (Rd):

$$V_{inst,d} \leq G_{stb,d} + R_d \qquad dove: \quad V_{inst,d} = G_{inst,d} + Q_{inst,d}$$

The partial actions are shown in Table 2.4 and are to be combined in an appropriate way with those relating to geotechnical parameters (M2).

Table 2.4 – Partial factors on actions for verification against the heaving of limit states.

LOADS	EFFECT	Partial coefficient γ_F (o γ_E)	Heaving (UPL)
Permanent	Favorable	γ_{G1}	0,9
	Unfavorable		1,1
Permanent not structural	Favorevole	γ_{G2}	0,0
	Unfavorable		1,5
Variable	Favorable	γ_{Qi}	0,0
	Unfavorable		1,5

The stability control is performed to siphoning verifying that the design value of the pore pressure instabilizzante ($u_{inst,\ d}$) prove no more than the design value of the total voltage stabilizer ($\sigma_{stb,\ d}$), taking into account the partial factors in Table 2.5:

Table 2.5 – *Partial factors on actions for verification against pyping serviceability limit states.*

LOADS	EFFECT	Partial coefficient γ_F (o γ_E)	Pyping (HYD)
Permanent	Favorable	γ_{G1}	0,9
	unfavorable		1,3
Permanent not structural	Favorable	γ_{G2}	0,0
	unfavorable		1,5
Variable	Favorable	γ_{Qi}	0,0
	unfavorable		1,5

2.3 INTRODUCTION TO SEISMIC ACTION

The definition of the design seismic putting into account the effects of local seismic response represents the element of greater novelty of the new technical regulations for construction than codified in the existing national seismic codes.

The Ministerial Decree of 16/01/1996 seismic areas classified into three categories (I, II and III) are characterized by different degrees of seismicity, which corresponded to the so-called seismic coefficients C, respectively 0.1, 0:07 and 0:04. This classification resulted largely from the macroseismic intensity maps, which in turn were based on the observation of the effects induced by earthquakes on the surface, the physical environment, on manufactured goods and people (such as the Mercalli scale).

In practice, the seismic classification, and the actions that congruence is determined, derived from an observation of the phenomenon of earthquake that could be called "top down" and "a posteriori" from above in the physical sense of the term, since it is observed the physical environment that is built, and in retrospect, since it took into account the effects produced at the end of the earthquake, according to the "danger" inherent in the site and the "vulnerability" of the physical and built.

The effects of local conditions were almost entirely neglected; was only reference in the definition of a coefficient of foundation ε: "It is assumed as a rule ε = 1. In the presence of stratigraphy characterized by alluvial deposits of variable thickness from 5 to 20 meters, overlying cohesive soils or lithoid with significantly higher mechanical properties, it is agreed by the coefficient ε the value 1.3. "This coefficient was therefore a kind of magic number , which was based solely on the nature of the deposit, and not on quantitative assessments of the real mechanical characteristics of soil.

The Ministerial Decree of 14/01/2008 according to the European standard, however, is completely different orientation in the evaluation of the seismicity of an area, because it springs from an observation of the seismic phenomenon that can be described as "bottom up" and "a priori" from below in the physical sense of the term, since it is looking directly at the seismic motion in its propagation from underground "deep" toward the free surface, and a priori, since it only takes into account the seismic zonation of seismic motion expected (in terms of acceleration), before it produces its effects on the physical and built.

Ultimately, the new standard is primarily designed to identify the value of a particular maximum acceleration at the end of the journey of the seismic motion from the area of origin (seismic source) up to the surface of a rigid formation outcropping.
Separately takes into account, in a more rational way, the presence of loose soil to covering the rigid formation, and therefore the so-called "local", through the identification of different classes of soil and topographical several categories.

2.3.1 UNDERGROUND CATEGORIES

Assuming that the subsoil of the site should be free from the risk of collapse phenomena (instability of slopes and liquefaction), the seismic action may be determined by reference to a simplified method based on the identification of categories of ground reference. Are briefly summarized in Table 2.6 and Figure 2.1.
They identified seven different types of subsoil; the first five identified by the letters A to E, and the other two as S1 and S2.
The classification thus identifies subsoils to stiffness gradually decreasing, starting from the subsoil type A, formed from virtually rock outcropping or covered by a layer less rigid of the maximum thickness of 5 m, up to subsoils S1 and S2, highly deformable and susceptible to phenomena breaking only for the seismic action.
For each of the first five types of subsoils will be defined a number of parameters characterizing the seismic motion at the surface, on the contrary for subsoils S1 and S2 is imposed to make specific studies for the determination of the seismic actions.
The most significant mechanical parameter for the characterization of the type of underground is the equivalent speed of the shear waves $V_{S,30}$ of the first 30 m of the subsoil, which is defined by the expression:

$$V_{S,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_{S,i}}} \left[m/s \right]$$

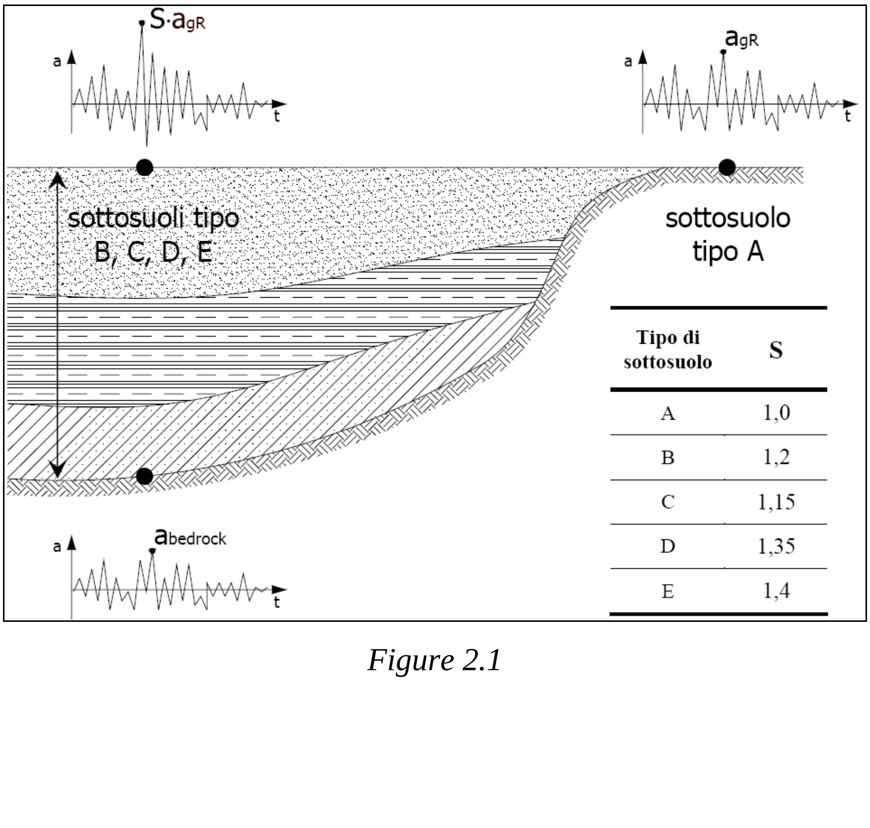
where $V_{S,i}$ is the speed of shear waves in the ith layer.
In the case of support works of natural soils, the depth of 30m is referred to the head of the work.
For retaining walls of embankments, that depth is instead referred to the plan sets the foundation.

The formulation, similar to that of the permeability in the series of stratified subsoils, favors the contribution of the layers more deformable, providing a speed equivalent $V_{S,30}$ significantly

conditioned by the lowest speed present in the first 30 m of the subsoil.
It does not take into account instead of the actual sequence of the layers, which is an additional element conditioning the seismic response (however difficult to contemplate in a diagram, as rational, however simplified subsoil).

Table 2.6 – *Subsoil category*

Category	Description
A	Outcropping rock masses or very stiff soils characterized by values of Vs, 30 greater than 800 m / s, optionally comprising a layer on the surface of alteration, with a maximum thickness of 3 m.
B	Soft rocks and deposits of coarse-grained soils or soils very densely packed fine grain very consistent with thicknesses greater than 30 m, characterized by a gradual improvement of the mechanical properties with depth and values of Vs, 30 between 360 m / s and 800 m / s (ie NSPT, 30> 50 coarse-grained soils and cu, 30> 250 kPa in fine-grained soils).
C	Deposits of coarse-grained soils average thickened or Fine grained soils consistent with average thicknesses greater than 30 m, characterized by a gradual improvement of the mechanical properties with depth and values of Vs, 30 between 180 m / s and 360 m / s (ie 15 <NSPT, 30 <50 coarse-grained soils and 70 <cu, 30 <250 kPa in fine-grained soils).
D	Deposits of coarse-grained soils or poorly thickened Fine grained soils poorly consistent with thicknesses greater than 30 m, characterized by a gradual improvement of the mechanical properties with depth and values of Vs, 30 less than 180 m / s (or NSPT, 30 <15 in the coarse-grained soils and cu, 30 <70 kPa in fine-grained soils).
E	Lots of subsoils of type C or D to a thickness not exceeding 20 m, placed on the reference substrate (with Vs> 800 m / s).



Tipo di sottosuolo	S
A	1,0
B	1,2
C	1,15
D	1,35
E	1,4

Figure 2.1

2.3.2 TOPOGRAPHIC CONDITIONS

For simple surface conditions can adopt the classification of Table 2.7 refers to geometric configurations predominantly two-dimensional, elongated ridges or crests, which must be considered in the definition of the seismic if greater height of 30 m.

Table 2.7 – *Categorie topographyc*

Category	Characteristics of the topography surface
T1	Flat surface, slopes and isolated peaks with an average inclination i ≤ 15 °
T2	Slopes with an average inclination i> 15 °
T3	Reliefs with crest width much smaller than the base and average slope of 15 ° ≤ i ≤ 30 °
T4	Reliefs with crest width much smaller than the base and average slope of 15 ° ≤ i ≤ 30 °

2.3.3 SEISMIC HAZARD

The seismic action on buildings is assessed, as mentioned earlier, starting from a "basic seismic hazard", in ideal conditions of the reference site rigid with the topographic surface horizontal (category A).

Above-mentioned "seismic hazard" is the primary element of knowledge for the determination of seismic actions and must be equipped with a sufficient level of detail in terms of geographical and temporal; these conditions can be considered fulfilled:

- In terms of values of maximum horizontal acceleration ag and parameters that define the response spectra in terms of rigid reference site;
- At the points of reference grid whose nodes are no more than 10km;
- For different probabilities of exceedance and different return periods TR.

The seismic action thus identified is then varied in the manner specified by the NTC, to take account of the changes produced by the local conditions of the subsurface stratigraphic actually present in the construction site and the surface morphology. These changes characterize the seismic response.

The availability of timely and detailed information so allows it to adopt in the design and verification of construction, better values of the seismic action related to the seismic hazard of the site, the nominal life of the building and the use for which it is intended, thus allowing solutions easy, meaningful and direct the design problem.

At present, the seismic hazard in the range of reference grid reference is provided by the data published on the website http://esse1.mi.ingv.it/.

2.3.4 THE DESIGN SEISMIC ACTION

The project actions are obtained in accordance with the NTC, 2008, from acceleration ag and related spectral shapes.

Spectral shapes are defined, on site reference horizontal rigid, in function of three parameters:

• a_g: maximum horizontal acceleration to the ground;

• F_0: maximum value of the amplification factor of the spectrum in horizontal acceleration

• * T_C: period beginning tract at a constant speed in horizontal acceleration spectrum.

The spectral shapes are characterized by the selected probability of exceedance P_{VR} in the period associated with the limit state under consideration and to the life of V_R reference to identify the seismic actions.

To this end it is convenient to use as a parameter characterizing the seismic hazard, the return period of the seismic T_R, expressed in years. Fixed life reference V_R, the two parameters T_R and P_{VR} are immediately expressible, one in function of the other, by the expression:

$$T_R = -\frac{V_R}{\ln(1 - P_{VR})}$$

The values of the parameters ag, F_0 and T_C * relative to the seismic hazard of reference grid in the target range are provided in the tables in Annex B of the NTC.

The grid points of reference are defined in terms of latitude and longitude and ordered the Latitude and Longitude increasing by varying the first and then the Latitude Longitude. The acceleration at the site a_g is expressed in g/9.81; F_0 is dimensionless, T_C * is expressed in seconds.

2.3.4.1 Limit states and relative exceedance probabilities

Against seismic actions limit states, both of exercise that last, are identified by referring to the performance of the construction as a whole, including the structural elements, those not structural and installations.

The limit states are:

• **Limit State Operation (SLO):**

Following the earthquake the building as a whole does not suffer significant harm;

• **Damage Limit State (SLD):**

Following the earthquake the building as a whole, suffers damage such as not to endanger users and immediately usable while still remaining in the interruption of a piece of equipment.

The ultimate limit states are:

• **Limit State for the Protection of Life (SLV):**

Following the earthquake the building suffered cracks and collapses of non-structural components and plant damage and significant structural component which is associated with a significant loss of stiffness against horizontal actions, the construction preserves instead a part of the strength and stiffness to vertical actions and a margin of safety against collapse due to horizontal seismic actions;

• **Limit State of collapse prevention (SLC):** Following the earthquake the building suffered serious fractures and collapse of the non-structural components and engineering and severe damage of structural components, the building still retains a margin of safety for vertical actions and a small margin of safety against collapse due to horizontal actions.

The probability of exceeding the reporting period PVR, which relate to locate the seismic action agent in each of the limit states considered are shown in the following Table 2.8.

Table2.8 – *Probability of exceeding PVR to vary the limit state considered*

Stati Limit state		**PVR: Probability of exceeding the reporting period VR**
effective limit state	SLO	81%
	SLD	63%
ultimate limit states	SLV	10%
	SLC	5%

2.3.4.2 Nominal life

The nominal life of V_N structural work is understood as the number of years in which the structure, provided that subject to routine maintenance, must be able to be used for the purpose for which it is intended. The nominal life of different types of works is defined in Table 2.9.

Table 2.9 – *Nominal life* V_N *for different type of structure*

TYPE OF CONSTRUCTION		**Nominal life V_N (years)**
1	Temporary works - Works provisional - Structures in the construction phase	≤ 10
2	Ordinary works, bridges, dams and infrastructure projects with small dimensions or importance	≥ 50
3	Great works, bridges, dams and infrastructure projects large or strategically important	≥ 100

2.3.4.3 Use Classes

In the presence of seismic activity, with reference to the consequences of a disruption of operations or of a possible collapse, the buildings are divided into use classes defined as follows:

• Class I: Buildings with only occasional presence of people, farm buildings.

• Class II: Building the use of which provides normal crowds, no content harmful to the environment and without essential public and social functions.

• Class III buildings the use of which provides significant crowding.

• Class IV: Buildings with public functions or strategic important, also with reference to the management of civil protection in the event of a disaster.

(for details, refer to Paragraph 2.4.2 of the DM 14/01/2008)

2.3.4.4 Reference Period

Seismic actions on each building are evaluated in relation to a reference period of V_R that is derived for each type of construction, multiplying the nominal life V_N by the coefficient of Use CU:

The value of the coefficient of use CU is defined, to vary the class of use, as shown in Table 2.10.

Table 2.10. – *values of* C_U

uses classes	I	II	III	IV
coefficient C_U	0,7	1,0	1,5	2,0

2.3.5 DETERMINATION OF SEISMIC ACTION

We want to determine the seismic action to be applied to a reinforced soil slope to be built in the town of Montagnana (Padova).

In this case, the work falls within the type of construction 2: "Works ordinary, bridges, dams and infrastructure projects of limited size or importance."

The nominal life is therefore V_N> 50 years.

The class is the use of Class II: "Construction the use of which provides normal crowds, no content harmful to the environment and without public and social functions."

The reference period for the seismic action V_R is given by:

$$V_R = V_N \cdot C_U = 50 \cdot 1 = 50 \quad years$$

The chances of overcoming P_{VR} in the reference period V_R, which relate to locate the seismic action , is equal to 10% in the case of the ultimate limit state SLV.

The return period of the seismic TR is calculated as:

$$T_R = -\frac{V_R}{\ln(1 - P_{VR})} = -\frac{50}{\ln(1 - 0{,}1)} = 475$$

The values of the parameters ag, F0 and TC * relative to the seismic hazard in the target range are provided in the tables (Annex B) of the NTC.

3. SOIL REINFORCEMENT PRINCIPLES

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for soil's lack of tensile resistance. The improved tensile properties are a result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:

• Stress transfer between the soil and reinforcement takes place continuously along the reinforcement.

• Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

Stress Transfer Mechanisms

Stresses are transferred between soil and reinforcement by friction (figure 1a) and/or passive resistance (figure 1b) depending on reinforcement geometry:

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile and some geogrid layers.

Passive resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is generally considered to be the primary interaction for rigid geogrids, bar mat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance.

The contribution of each transfer mechanism for a particular reinforcement will depend on the roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are the soil characteristics, including grain size, grain size distribution, particle shape, density, water content, cohesion, and stiffness.

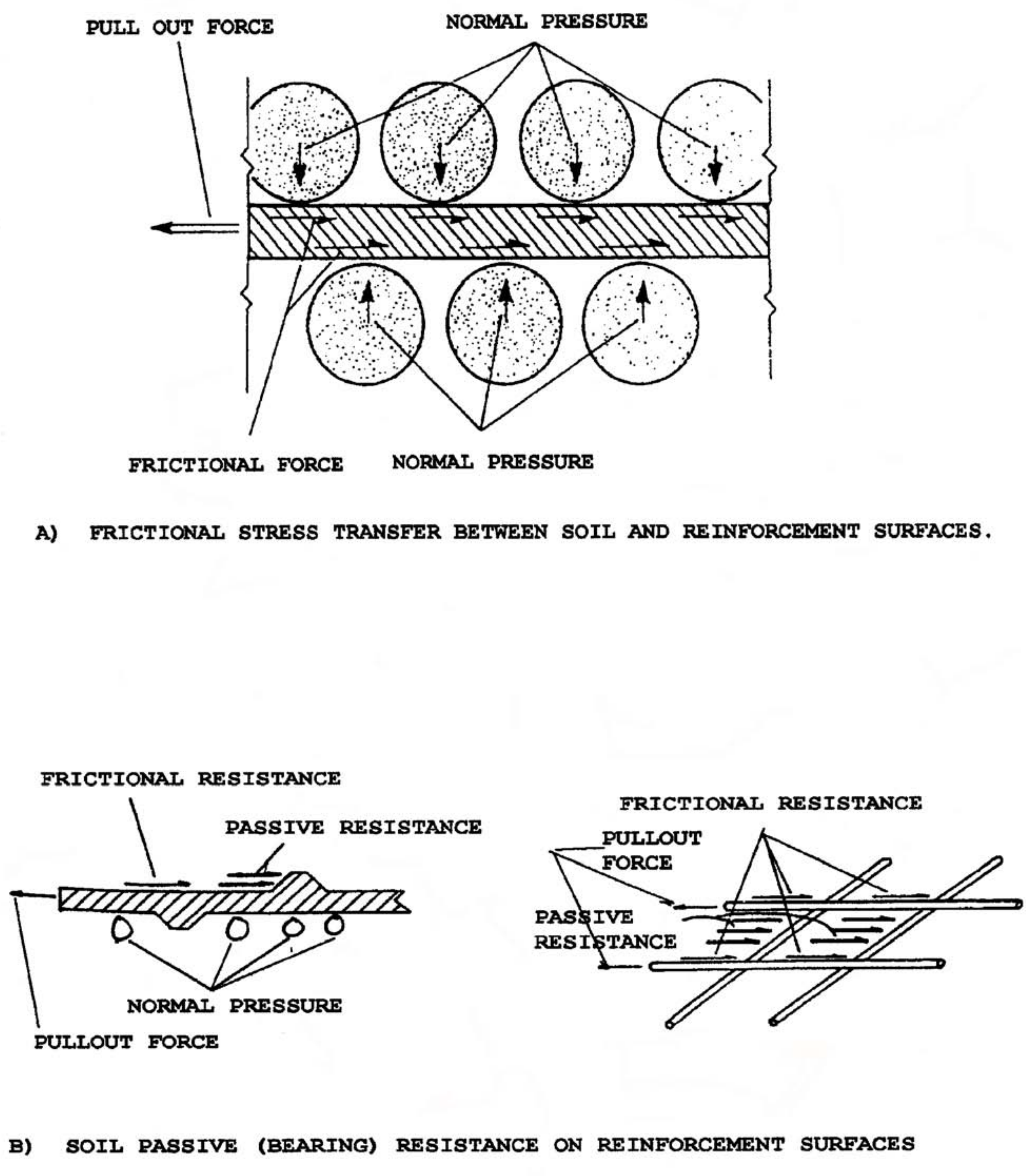


Figure 1. Stress transfer mechanisms for soil reinforcement.

Mode of Reinforcement Action
The primary function of reinforcements is to restrain soil deformations. In so doing, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

Tension is the most common mode of action of tensile reinforcements. All "longitudinal" reinforcing elements (i.e., reinforcing elements aligned in the direction of soil extension) are generally subjected to high tensile stresses. Tensile stresses are also developed in flexible reinforcements that cross shear planes.

Shear and Bending. "Transverse" reinforcing elements that have some rigidity, can withstand shear stress and bending moments.

3.1 Soil reinforcement interaction using normalized concepts

Soil-interaction (pullout capacity) coefficients have been developed by laboratory and field studies, using a number of different approaches, methods, and evaluation criteria. A unified normalized approach has been recently developed, and is detailed below.

a. Evaluation of Pullout Performance

The design of the soil reinforcement system requires an evaluation of the long-term pullout performance with respect to three basic criteria:

- Pullout capacity, i.e., the pullout resistance of each reinforcement should be adequate to resist the design working tensile force in the reinforcement with a specified factor of safety.

- Allowable displacement, i.e., the relative soil-to-reinforcement displacement required to mobilize the design tensile force should be smaller than the allowable displacement.

- Long-term displacement, i.e., the pullout load should be smaller than the critical creep load.

The pullout resistance of the reinforcement is mobilized through one or a combination of the two basic soil-reinforcement interaction mechanisms, i.e., interface friction and passive soil resistance against transverse elements of composite reinforcements such as bar mats, wire meshes, or geogrids.

The load transfer mechanisms mobilized by a specific reinforcement depends primarily upon its structural geometry (i.e., composite reinforcement such as grids, versus linear or planar elements, thickness of transverse elements, and aperture dimension). The soil-to-reinforcement relative movement required to mobilize the design tensile force depends mainly upon the load transfer mechanism, the extensibility of the reinforcement material, the soil type, and confining pressure.

The long-term pullout performance (i.e., displacement under constant design load) is predominantly controlled by the creep characteristics of the soil and the reinforcement material. Soil reinforcement systems will generally not be used with cohesive soils susceptible to creep. Therefore, creep is primarily an issue of the type of reinforcement. Table 4 provides, for generic reinforcement types, the basic aspects of pullout performance in terms of the main load transfer mechanism, relative soil-to-reinforcement displacement required to fully mobilize the pullout resistance, and creep potential of the reinforcement in granular (and low plasticity cohesive) soils.

Table 4. Basic aspects of reinforcement pullout performance in granular and cohesive soils of low plasticity.

Generic Reinforcement Type	Major Load Transfer Mechanism	Range of Displacement at Specimen Front	Long Term Deformation
Inextensible strips			
smooth	Frictional	1.2 mm	Noncreeping
ribbed	Frictional + passive	12 mm	
Extensible composite plastic strips	Frictional	Dependent on reinforcement extensibility	Dependent on reinforcement structure and polymer creep
Extensible sheets			
geotextiles	Frictional	Dependent on reinforcement extensibility (25 to 100 mm)	Dependent on reinforcement structure and polymer creep characteristics
Inextensible grids			
bar mats	Passive + frictional	12 to 50 mm	Noncreeping
welded wire meshes	Frictional + passive	12 to 50 mm	Noncreeping
Extensible grids			
geogrids	Frictional +passive	Dependent on extensibility (25 to 50 mm)	Dependent on reinforcement structure and polymer creep characteristics
woven wire meshes	Frictional +passive	25 to 50 mm	Noncreeping

For geosynthetic (i.e., geogrid and geotextile) sheet reinforcement, the pullout resistance is based on a reduction in the available soil friction with the reduction factor often referred to as an Interaction Factor, C_i.

For RSS structures, the φ angle of the reinforced backfill is
normally established by test, as a reasonably wide range of backfills can be used. A lower bound value of 28 degrees is often used.

b. Interface Shear

The interface shear between sheet type geosynthetics (geotextiles, geogrids and geocomposite drains) and the soil is often lower than the friction angle of the soil itself and can form a slip plane. Therefore the interface friction coefficient tan ρ must be determined in order to evaluate sliding along the geosynthetic interface with the reinforced fill and, if appropriate, the foundation or retained fill soil.

The interface friction angle ρ is determined from soilgeosynthetic direct shear tests. In the absence of test results, the interface friction coefficient can be conservatively taken as b tan φ for geotextiles, geogrids and geonet type drainage composites. Other geosynthetics such as geomembranes and some geocomposite drain cores may have much lower interface values and tests should accordingly be performed.

3.2 Establishment of structural design properties

The structural design properties of reinforcement materials are a function of geometric characteristics, strength and stiffness, durability, and material type. The two most commonly used reinforcement materials, steel and geosynthetics, must be considered separately as follows:

3.2.1. Geometric Characteristics

Two types can be considered:

• **Strips, bars, and steel grids**. A layer of steel strips, bars, or grids is characterized by the cross-sectional area, the thickness and perimeter of the reinforcement element, and the center-to-center horizontal distance between elements (for steel grids, an element is considered to be a longitudinal member of the grid that extends into the wall).

• **Geotextiles and geogrids**. A layer of geosynthetic strips is characterized by the width of the strips and the center-to-center horizontal distance between them. The cross-sectional area is not needed, since the strength of a geosynthetic strip is expressed by a tensile force per unit width, rather than by stress. Difficulties in measuring the thickness of these thin and relatively compressible materials preclude reliable estimates of stress.

3.2.2. Strength Properties

Steel Reinforcement

For steel reinforcements, the design life is achieved by reducing the cross-sectional area of the reinforcement used in design calculations by the anticipated corrosion losses over the design life period as follows:

$$E_c = E_n - E_R$$

where E_c is the thickness of the reinforcement at the end of the design life, E_n the nominal thickness at construction, and E_R the sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure.

Geosynthetic Reinforcement

Geosynthetic reinforcement is more complex than for steel. The tensile properties of geosynthetics are affected by environmental factors such as creep, installation damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer can vary widely, and the details of polymer behavior for in-ground use are not completely understood. Ideally, T_a should be determined by thorough consideration of allowable elongation, creep potential and all possible strength degradation mechanisms.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking depending on polymer type. In addition, these materials are susceptible to installation damage and the effects of high temperature at the facing and connections. Temperatures can be as high as 50^{o}C compared with the normal range of in-ground temperature of 12^{o}C in cold and temperate climates to 30^{o}C in arid desert climates.

Degradation most commonly occurs from mechanical damage, long-term time dependent degradation caused by stress (creep), deterioration from exposure to ultraviolet light, and chemical or biological interaction with the surrounding environment. Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical and biological agents. Therefore, each product must be investigated individually.

Typically, polyester products (PET) are susceptible to aging strength reductions due to hydrolysis (water availability) and high temperatures. Hydrolysis and fiber dissolution are accelerated in alkaline regimes, below or near piezometric water levels or in areas of substantial rainfall where surface water percolation or capillary action ensures water availability over most of the year.

Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen) and or high temperatures. The level of oxygen in reinforced fills is a function of soil porosity, ground water location and other factors, and has been found to be slightly less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in the ground may proceed at an equal rate than those used above ground. Oxidation is accelerated by the presence of transition metals (Fe, Cu, Mn, Co, Cr) in the backfill as found in acid sulphate soils, slag fills, other industrial wastes or mine tailings containing transition metals. It should be noted that the resistance of polyolefin geosynthetics to oxidation is primarily a function of the proprietary antioxidant package added to the base resin, which differs for each product brand, even when formulated with the same base resin.

Most geosynthetic reinforcement is buried, and therefore ultraviolet (UV) stability is only of concern during construction and when the geosynthetic is used to wrap the wall or slope face. If used in exposed locations, the geosynthetic should be protected with coatings or facing units to prevent deterioration. Vegetative covers can also be considered in the case of open weave geotextiles or geogrids. Thick geosynthetics with ultraviolet stabilizers can be left exposed for several years or more without protection; however, long-term maintenance should be anticipated because of both UV deterioration and possible vandalism.

Damage during handling and construction, such as from abrasion and wear, punching and tear or scratching, notching, and cracking may occur in brittle polymer grids. These types of damage can only be avoided by care during handling and construction. Track type construction equipment should not travel directly on geosynthetic materials.

Polyester geosynthetics

PET geosynthetics are recommended for use in environments characterized by $3 < pH < 9$, only. The following reduction factors for PET aging (RF_D) are presently indicated for a 100 year design life in the absence of product specific testing.

Polyolefin geosynthetics

To mitigate thermal and oxidative degradative processes, polyolefin products are stabilized by the addition of antioxidants for both processing stability and long term functional stability. These antioxidant packages are proprietary to each manufacturer and their type, quantity and effectiveness varies. Without residual antioxidant protection (after processing), PP products are vulnerable to oxidation and significant strength loss within a projected 75 to 100 year design life at 20°C. Current data suggests that unstabilized PP has a half life of less than 50 years.

Therefore the anticipated functional life of a PP geosynthetic is to a great extent a function of the type and remaining antioxidant levels, and the rate of subsequent antioxidant consumption. Antioxidant consumption is related to the oxygen content in the ground, which in fills is only slightly less than atmospheric.

At present, heat aging protocols for PP products, at full or reduced atmospheric oxygen, with subsequent numerical analysis are available for PP products which exhibit no initial cracks or crazes in their as manufactured state, typically monofilaments. For PP products with initial crazes or cracks, typically tape products, or HDPE, heat aging testing protocols may change the nature of the product and therefore may lead to erroneous results. Alternate testing protocols using oxygen pressure as a time accelerator are under study and development.

3.4 General informations on the calculations of reinforced soil

The calculation of reinforced earth involves a series of tests in order to ascertain the loss of equilibrium of the structure (reinforced earth).
The tests can be divided into internal, external and inspections checks.

3.4.1 Internal checks

Internal audits relate to the determination of not overcoming resistances given by reinforcements for the mechanisms:
- Direct slide;
- Pull-out;
-Tensile strength.

The verification on direc slide consist to give a determinated length of reinforcement that prevent the sliding of the block of reinforced earth above the reinforcement itself. the resistance creep along a reinforcing element is given by the following expression:

$$\tau_{scor} = \sigma_v^{'} \cdot f_{ds} \cdot tg(\varphi^{'})$$

fds = direct slide coefficient.

In terms of strength, we have:

$$T_{scor} = L_{scor} \cdot B \cdot \tau_{scor}$$

Lscor = length of reinforcement.
B = reinforcement width.
The reinforcement length as to be for $T_{scor} > S$ with S thrust agent at the level of reinforcement.
The checks is satisfied if:

$$\frac{T_{scor}}{S} \geq FS_{scor}$$

The pull-out verification is to ensurethe length of the reinforcing enough to prevent the slippage of the reinforcement from the reinforced soil. The resistance to pull-out along a reinforcing element is given by the following expression:

$$\tau_{sfil} = \sigma_v^{'} \cdot f_{po} \cdot tg(\varphi^{'})$$

f_{po} = pull-out coefficient
In terms of strength we have:

$$T_{sfil} = L_{sfil} \cdot B \cdot 2 \cdot \tau_{sfil}$$

L_{sfil} = reinforcement length
B = reinforcement width.
The reinforcement length as to be for $T_{sfil} > S$ with S thrust agent at the level of reinforcement.
The checks is satisfied if:

$$\frac{T_{sfil}}{S} \geq FS_{sfil} \cdot$$

The tensile test is to ensure that the tension in the reinforcement does not exceed the admissible. The test is satisfied if:

$$\frac{P}{S} \geq FS_{traz}$$

P : Permissible resistance used for sizing;

$$P = \frac{LTDS}{FS_{giunzione} \cdot FS_{chimico} \cdot FS_{biologico} \cdot FS_{danni\ ambientali}}$$

LTDS : long-term resistance project;

3.4.2 External checks

The external checks determine the state of limit equilibrium of reinforced earth (seen as a rigid body and without the presence of reinforcements) for the following kinematics:
- Horizontal translation (sliding of the reinforced earth)
- Vertical translation (limit load of the complex reinforced earth-ground)
- Rotation (reversal of reinforced earth)
- Global equilibrium limit (overall stability of reinforced earth-surrounding land).

3.4.3 Combine checks

This checks regarding the search of the kinematic breaking covering the whole land reinforcements. The program analyzes the families of circular surfaces.
The calculation of the safety factor of the circular surface is carried out by the strips method taking into account the contribution of the resistance of reinforcements.

3.5 Earth thrust

The main problem in the calculation of a retaining wall is the determination of the thrust that the embankment exerts on the wall itself.

The foundations of the classical theory of earth pressure were posed in 1773 by Coulomb. After that Poncelet studies in 1840 and the theory of the boulder unlimited by Rankine. The theories of Coulomb and Rankine and the others derived from them are still today the most used for the calculation of the retaining walls.

Among the methods of calculation derived from the theory of Coulomb particular importance the Culmann method and the method of wedge attempt particularly suitable for implementation on a computer. Other theories based on the theory of plasticity such as that of Rosenfarb and Chen (1972) are still under-used and thus lack of reliable experimental results.
An analysis should take into account soil-structure interaction. In practice methods are used approximations that are based on the limit equilibrium method globally.

The most common methods are the method of Rankine and Coulomb (and derivatives).
It is assumed that the horizontal pressure that the ground exerts on the wall is related to the vertical pressure (hydrostatic) by a relation of the $\sigma h = k^* \sigma v$ where K is the coefficient of thrust.

The thrust coefficient is related to the type and extent of the displacement that the work itself suffers.
If the work does not have displacements,K coincides with the thrust coefficient Ko. The two methods mentioned above assume instead that the wall has a shift. In this case the thrust coefficient is reduced by the value Ko to Ka value (coefficient of active earth pressure).

3.5.1 RANKINE THEORY

The Rankine theory or boulder unlimited considers the soil in a state of equilibrium and supposes that there is no friction between the wall and the ground.
Considering the case of a ground incoerent with friction angle ϕ and said β the angle that the soil upstream of the wall forms with the horizontal, the coefficient of active thrust Ka is given by:

$$K_a = \cos\beta \quad \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

and then the lateral pressure, to a generic depth z, and the all thrust on the wall of height H is:

$$\sigma_a = \gamma\, z\, K_a$$

$$S = \int_0^H \gamma\, z\, K_a\, dz = \frac{1}{2}\gamma\, H^2 K_a$$

The thrust thus determined is inclined by an angle equal to β respect to the horizontal.

3.5.2 COULOMB THEORY

The Coulomb theory considers the hypothesis of a wedge thrust upstream of the wall that moves rigidly along a rupture rectilinear surface. From the balance of the thrust wedge is obtained that the ground exerts on the work. In particular Coulomb admits, contrary to the theory of Rankine, the existence of friction between the ground and the facing of the wall, and then the line of thrust is inclined respect to the normal to the facing of a same friction angle earth-wall.
The expression of the thrust exerted by an embankment, of specific weight γ, on a wall of height H, is expressed according to the theory of Coulomb by the following relationship:

$$S = 1/2 \quad \square \; H^2 \, Ka$$

Ka is the coefficient of active earth pressure of Coulomb in the revised version by Muller-Breslau, expressed as:

$$K_a = \frac{sin^2(\alpha + \phi)}{sin^2\alpha \times sin(\alpha - \delta) \times \left[1 + \frac{\sqrt{sin(\phi + \delta) sin(\phi - \beta)}}{\sqrt{sin(\alpha - \delta) sin(\alpha + \beta)}}\right]^2}$$

where ϕ is the angle of friction of the soil, α is the angle that the wall forms with the horizontal (α = 90 ° to the vertical wall), δ is the friction angle of the ground-wall, β is the slope of the embankment to the horizontal.
The thrust is inclined angle of friction ground/wall δ respect to the normal to the wall.
In both cases, the diagram of the pressures on the wall of the earth is triangular with the apex above.
The point of application of the thrust is located at the centroid of the diagram of pressures (1/3 H with respect to the base of the wall). Note that the expression of Ka loses meaning for β> ϕ. This coincides with what are often quite physically: the slope of the ground behind the wall can not exceed the angle of repose of the soil.
In the case in which the embankment is burdened by an overload uniform Q the expression of the pressure andnthrust become:

$$\sigma_a = (\gamma \; z + Q) K_a$$

$$S = \int_0^H (\gamma \; z + Q) K_a \; dz = (\frac{1}{2} \gamma \; H^2 + QH) K_a$$

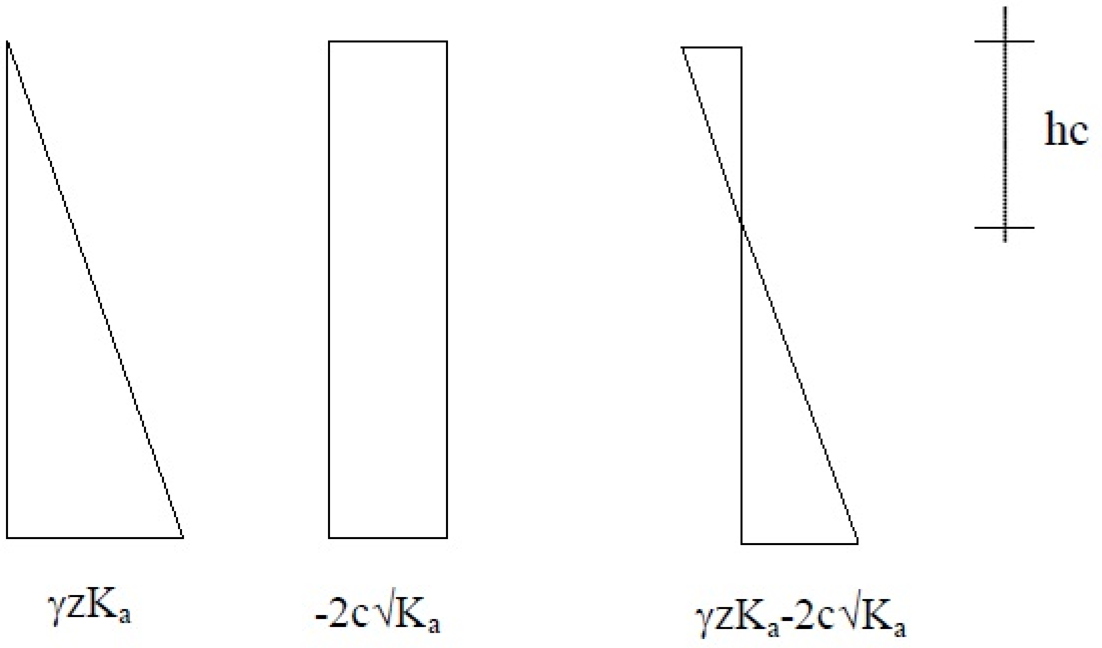
To the load Q correspond a diagram of the pressure applied to the resulting rectangular 1/2H.
Both methods examined consider a soil without cohesion.
In the case of soil with cohesion c the expression of the pressure exerted on the wall, to generic depth z, becomes:

$$\sigma_a = \gamma \; z K_a - 2c\sqrt{K_a}$$

At the triangular diagram, expressed by the term γ z Ka, you subtract the rectangular diagram linked to term cohesion. The pressure σa is negative for values of z less than:

$$h_c = \frac{2c}{\gamma \sqrt{K_a}}$$

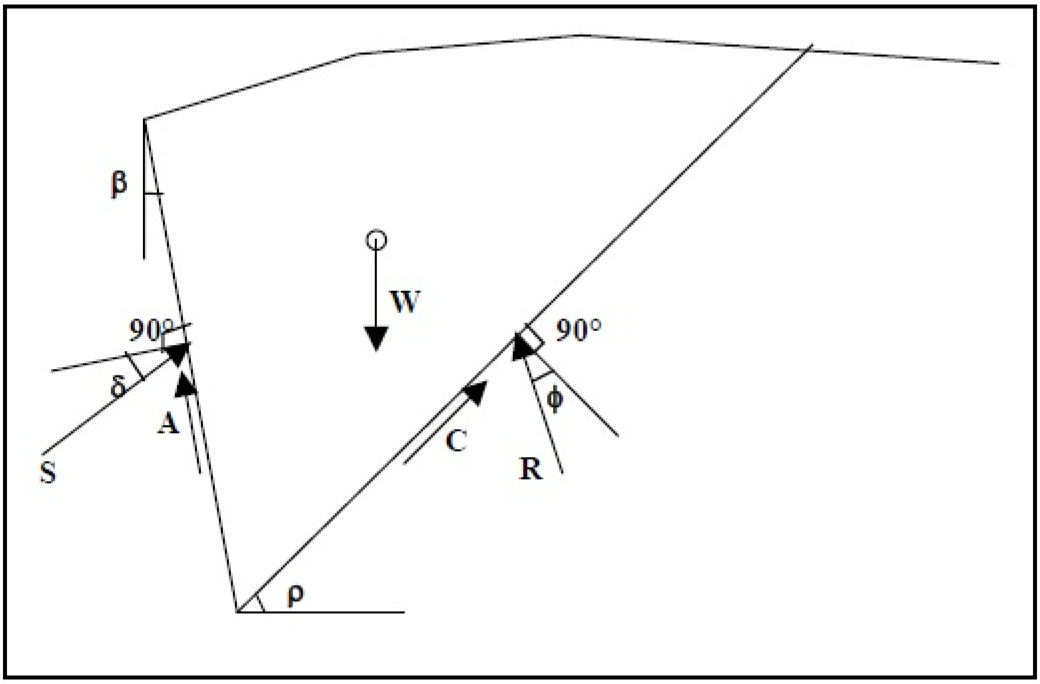
hc is called critical height and represents the depth of potential fracture of the soil. It is clear that if the height of the wall is less than hc have no thrust on the wall.
In the case of soil with cohesion the program deletes (a safety advantage) the part of diagram with negative pressure. Then deletes the diagram that extends from the top Wall to depth z = hc.



3.5.3 COULMANN Theory

The Culmann method adopts the same basic assumptions of the Coulomb method. the difference is that whileCoulomb considers an embankment with constant slope surface and load uniformly distributed (which allows to obtain an expression in the form closed for the value of the thrust) the Culmann method allows the analysis situations with generic form profile andboth concentrated and distributed loads however they are put.

Furthermore, compared to methods previously treated, it is more immediate and linear take into account the cohesion of the rock pushing. The method of Culmann, born as a method essentially graph, has evolved to be treated by numerical analysis (known in this form as a method of wedge attempt).



Like the two previous methods, this method also considers a failure surface flat.
The steps of resolving procedure are the following:
it imposes a rupture surface (inclined by an angle ρ respect to the horizontal) and considering the wedge thrust delimited by the surface of rupture itself, from the wall on which it calculates the thrust, and from the soil profile;

evaluating all the forces acting on the wedge thrust namely weight (W), loads acting on the the ground surface, frictional resistance (R) and cohesion (C) along the surface of rupture and resistance to cohesion along the wall (A);
from the equations of equilibrium is obtained the value of the push S on the wall (inclined angle of friction soil - δ wall with respect to the normal to the wall).

This process is iterated to find the angle of rupture to which the push is highest.
The convergence is not achieved if the embankment is tilted at an angle greater than the angle friction of the land (see the remarks made for methods of Coulomb and Rankine).

In cases where it is applicable the Coulomb method (upstream rectilinear profile and load evenly distributed) the results obtained by the Culmann method coincide with those of the Coulomb method.

The method, as it has been described, does not allow to obtain the diagram of the pressures agent on the wall (and hence the stresses along the wall) and is also difficult to determine the point of application of the thrust.

In the program the process has been implemented in different ways.
It breaks down the wall height in many sections of width dz.
In correspondence of each ordinate zi is the wedge of rupture and the thrust is getting the distribution of the thrust S (z) along the height of the wall.

Note the distribution of the thrusts along the height of the wall, the pressure to a generic depth z, with respect to the top of the wall, is expressed by:

$$\sigma(z) = \frac{dS}{dz}$$

With the diagram of the pressures is possible to obtain the point of application of the thrust. also from pressure diagram is easy to derive the evolution of the stresses along the wall, with the usual methods of building science.

3.5.4 MONONOBE – OKABE Theory

The Mononobe-Okabe method adopts the same assumptions of the theory of Coulomb wedge thrust upstream of the wall that moves rigidly along a rectilinear rupture surface. Take into account also the inertia seismic wedge in horizontal and vertical direction. From the balance of the wedge is obtained the pressure that the ground exerts on the work of support in seismic conditions.

It is reckoned, as in the theory of Coulomb, the existence of the friction between the soil and the facing of the wall, and then the straight thrust is inclined with respect to the normal to the facing of a same friction angle soil-wall.

The expression of the total thrust (more static seismic) exerted by an embankment, weight volume γ, on a wall of height H, is expressed according to the theory of Mononobe-Okabe by the following relationship:

$$S = 1/2(1 \pm k_v)\gamma H^2 K_a$$

Ka rapresent the coefficient of active thrust.

$$K_a = \frac{\sin(\alpha + \phi - \theta)}{\sin^2\alpha \, \sin(\alpha - \delta - \theta) * \left[1 + \frac{\sqrt{\sin(\phi + \delta)\sin(\phi - \beta - \theta)}}{\sqrt{\sin(\alpha - \delta - \theta)\sin(\alpha + \beta)}}\right]}$$

where β is the angle of inclination of the embankment and α the angle of inclination of the wall compared to the
vertical.
The angle θ is linked to the seismic coefficient by the following expression

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

where kh and kv represent coefficient of seismic intensity in horizontal and vertical.
In case the value of the non-seismic thrust coefficient of the method of Mononobe-Okabe coincides with value given by the ratio of Coulomb.

3.6 Seismic actions

To take into account the increase in thrust due to the earthquake, reference is made to the Mononobe- Okabe method (referred to by the Italian legislation).
The Italian legislation suggests taking into account an increase in thrust due to the earthquake as following.
Said ε the inclination of the embankment β respect to the horizontal and the inclination of the wall compared to the vertical, calculate the thrust S 'considering a slope of the embankment wall and equal to:

$$\varepsilon' = \varepsilon + \theta$$
$$\beta' = \beta + \theta$$

where θ = arctg (C) where C is the coefficient of seismic intensity
If one adopts the Ordinance 2003, the expression of θ is the following:

$$\theta = \mathrm{arctg}\frac{k_h}{1 \pm k_v}$$

being the seismic coefficient kh horizontal and vertical seismic coefficient kv, defined in terms of kh.
In the presence of water upstream, θ assumes the following expressions:

Soil with low permeability

$$\theta = \mathrm{arctg}\left(\frac{\gamma_{sat}}{\gamma_{sat} - \gamma_w} * \frac{k_h}{1 \pm k_v}\right)$$

Soil whit high permeability

$$\theta = \mathrm{arctg}\left(\frac{\gamma}{\gamma_{sat} - \gamma_w} * \frac{k_h}{1 \pm k_v}\right)$$

Said S thrust calculated in static conditions the increase of thrust to be applied is expressed by:

$$\Delta S = AS' - S$$

Where A is :

$$A = \frac{\cos^2(\beta + \theta)}{\cos^2 \beta \cos \theta}$$

Adopting the method of Okabe Mononobe-for the calculation of the thrust, the coefficient A is set equal to 1.

If you adopt the legislation in 1988 that increased thrust is applied at a distance from the base equal to 2/3 of the wall height of thrust.

If you adopt Ordinance 2003, this increase in thrust is applied at mid-height of the wall thrust in the case of rectangular shape of the diagram of seismic increase, at the same point where acts the static thrust in the case where the shape of the diagram is equal to the increase in seismic the static diagram.

In addition to this increase must take account of the forces of inertia that are awakened for effect of earthquake.
If you adopt the legislation in 1988, the horizontal force of inertia is evaluated as

$$F_i = CW$$

If one adopts the Ordinance 2003 horizontal and vertical inertia forces have the following expressions

$$F_{iH} = k_h W \qquad F_{iV} = \pm k_v W$$

W is the weight of the wall,plus the ground above the foundation upstream and damage from overcharging. These forces must be applied in the center of gravity of the weights. The method of Culmann automatically takes into account the increase of thrust. just insert in the equation the inertia of the wedge thrust. The failure surface in the event of an earthquake is less inclined, with respect to the horizontal, of the corresponding surface in the absence of an earthquake.

3.7 Thrust in presence of water

In the event that upstream of the wall is present the aquifer the diagram of the pressures on the wall is modified because of sottospinta that the water exerts on the ground. The weight of the volume of soil to above the line of groundwater does not change. Conversely below the groundwater level must be considered the weight of volume of waterline:

$$\gamma_a = \gamma_{sat} - \gamma_w$$

where γ sat is the volume weight of the saturated soil (depending from the index of the pores) and γw is the specific weight water. So the diagram of the pressures below the line groundwater has a slope smaller.
The diagram thus obtained must be added the triangular diagram related to the hydrostatic pressure exerted by the water.

3.8 Overturning checks

The overturning check consists in determining the resultant moment of all the forces that tend to Do overturn the wall (overturning moment Mr) and the resulting moment of all the forces that tend to stabilize the wall (stabilizing moment Ms) compared to the downstream edge of the foundation and verify that the ratio Ms / Mr is greater than a given safety factor η.
The Italian legislation requires that both $\eta \geq 1.5$.
It must therefore be tested the following inequality:

$$\frac{M_s}{M_r} \geq 1.5$$

The overturning moment Mr is given by the horizontal component of the thrust S, by the forces of inertia of the wall and soil charged on the foundation of the mountain (presence of the earthquake) for the respective arms. In stabilizing moment intervenes the weight of the wall (applied in the center of gravity) and the weight of the soil imposed on the foundation of the mountain. As for the vertical component of thrust it will be stabilizing if the earth-wall friction angle is positive δ, δ overturning if it is negative.

δ is positive when the embankment that slides with respect to the wall, is negative when the wall which tends to slide with respect to the embankment (this may be the case of a shoulder as a bridge burdened by loads significant).

3.8 Sliding chacks

For the sliding check the wall along the foundation plan must show that the sum of all the forces parallel to the laying surface, which tend to slide the wall must be less than all the forces parallel to the sliding plane, which are opposed to slipping, according to a certain safety factor.

In particular, the Italian legislation requires that the relationship between the resultant forces resistant to slipping and Fr the resultant of the forces that tend to slide the wall Fs:

$$\frac{F_r}{F_s} \geq 1.3$$

The forces involved in the Fs are: the component of the thrust parallel to the plane of the foundation and the component of the forces of inertia parallel to the plane of the foundation. The resisting force is given by the friction resistance and the resistance to adhesion along the base of foundation.

That the N component normal to the plane of the founding of the total axle load in foundation and pointing with δ the friction angle soil-foundation, with about membership terrenofondazione Br and the width of the foundation reagent, the resistant force can be expressed as Fr = Ntg□ + caBr.

The legislation allows you to compute, in the opposing forces, an aliquot of any passive thrust due to the ground situated downstream of the wall. In this case, however, the safety factor must be appropriately increased. The rate of passive thrust that can be considered for the purposes of verificationshift may not exceed 50%.

Regarding the angle of friction-earth foundation, δ, several authors suggest to assume a δ value comprised between 2/3 of the angle of friction of the ground, and the value of the angle friction of the ground.

3.10 Check the limit load of the whole foundation - soil

The relationship between the ultimate load in the foundation and the resultant of the loads induced by the wall on the ground of foundation must be greater than 2.

With regard to the determination of the ultimate load in the foundation see te the following chapter. Q said in the load on the foundation and the load qult last in the foundation, must be

$$q_{ult}/q > 2$$

The program determines the maximum and minimum tension on the foundation according to the report the bending section for non-reactive strain. If the center of pressure is internal to the central core of inertia of the footprint of the foundation of the section is all reagent and the maximum and minimum values are given by:

$$\sigma_t = \frac{N}{A} \pm \frac{M}{I} y$$

where A and I are respectively the area and the inertia of the section of footprint and y is the distance fromcenter of gravity of the edges, N and M are the normal stress and the moment with respect to the center of gravity. In the case of section of rectangular footprint (size BxL) y = L / 2.

If the center of pressure is external to the core section is partially divided. In this case, such and the eccentricity of the load center of gravity with respect to u and the distance from the center of pressure with respect to the edge more compressed (u = L/2-e) the maximum voltage is given by:

$$\sigma_t = \frac{2N}{3uB}$$

In the case of non-rectangular section of footprint (see the case of wall with buttresses having foundation protruding from the foundation wall) research of the neutral axis must be done by trial and error solving the well-known cubic equation of buckling.

3.10.1 Calculation of the limit load

The foundation soil of any structure must be able to support the load that is transmitted by the overlying structures without encountering breakage and without that the sagging of structure are excessive. In this chapter we discuss the problem of determining the shear strength limit (ultimate load or the load limit) of a foundation. Will propose the solutions obtained by different authors (Terzaghi, Meyerhof, Hansen, Vesic) and used by the program.

All formulas proposals have trinomia a form in which each term is related to the cohesion, the friction angle and specific gravity. They differ in the introduction of correction factors for take into account the depth of the foundation, the eccentricity and inclination of the load, etc.. In the writing of the various formulas will be used the following symbolism:

c Cohesion
Ca Adhesion along the base of the foundation ($ca \leq c$)
ϕ angle of friction
δ foundation soil friction angle

γ Specific weight of soil
Kp Coefficient of passive thrust expressed by $Kp = \tan^2 (45° + \phi / 2)$
B Width of the foundation
L The length of the foundation
D Depth of the laying of the foundation
p geostatic pressure p at the laying of the floor foundation
qult last load of the foundation

3.10.2 TERZAGHI

Terzaghi proposed the following expression for the calculation of the bearing capacity of a foundation superficial

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

where the factors Nc, Nq, N γ are expressed by the relations:

$$N_q = \frac{e^{2(0.75\pi - \phi/2)\ tg(\phi)}}{2\cos^2(45 + \phi/2)}$$

$$N_c = (N_q - 1)\ ctg\phi$$

$$N_\gamma = \frac{tg\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

The form factors s_γ and sc that appear in the expression of qult depend on the shape of the foundation.
In particular value 1 or elongated rectangular ribbon for foundations and are respectively 1.3 and 0.8 for square foundations.

Regarding the value of N γ it depends on the factor Kp γ referred Terzaghi not leave no wording analytical. Several authors recommend using, for N γ, rather than the expression provided by Terzaghi formulations obtained by other authors (Vesic, Spangler and Handy). Terzaghi's formula is valid for shallow foundations with D $\leq$ B and takes no account of inclination and eccentricity and inclination of the foundation of the load.

3.10.3 MEYERHOF

Meyerhof proposes for the calculation of the bearing capacity the following expression:

vertical load $$q_{ult} = c\, N_c\, s_c\, d_c + q\, N_q\, s_q\, d_q + 0.5\, B\, \gamma\, N_\gamma\, s_\gamma\, d_\gamma$$

inclined load $$q_{ult} = c\, N_c\, i_c\, d_c + q\, N_q\, i_q\, d_q + 0.5\, B\, \gamma\, N_\gamma\, i_\gamma\, d_\gamma$$

where dc, dq, d γ, are the factors of depth, sc, sq, s γ, are the form factors and ic, iq, the γ, are inclination factors of the load.

The factors Nc, Nq, N γ are given by the following expressions:

$$N_q = e^{\pi\, tg\ \phi}\, K_p$$
$$N_c = \left(N_q - 1\right)\, ctg\phi$$
$$N_\gamma = \left(N_q - 1\right)\, tg\left(1.4\phi\right)$$

To form factors sc, sq, s γ, depth dc, dq, d γ, and tilt ic, iq, the γ, we have:

$$s_c = 1 + 0.2 K_p \frac{B}{L}$$

ϕ = 0 $\quad s_q = s_\gamma = 1$

ϕ > 0 $\quad s_q = s_\gamma = 1 + 0.1 K_p \frac{B}{L}$

$$d_c = 1 + 0.2\sqrt{K_p}\, \frac{D}{B}$$

ϕ = 0 $\quad d_q = d_\gamma = 1$

ϕ > 0 $\quad d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\, \frac{D}{B}$

$$i_c = i_q = \left(1 - \frac{\vartheta}{90}\right)^2$$

ϕ = 0 $\quad i_\gamma = 0$

ϕ > 0 $\quad i_\gamma = \left(1 - \frac{\vartheta}{\phi}\right)^2$

In expressions of the factors of inclination θ is the angle that the straight line of action of the load form with the vertical.

The values of qult that are obtained by the formula of Meyerhof are comparable to those obtained using the formula of Terzaghi for low values of the ratio D / B. The difference is accentuated when the ratio D / B becomes higher.

3.10.4 HANSEN

The expressions of Hansen for the calculation of the bearing capacity will differ depending on whether it is to presence of a purely cohesive soil (ϕ = 0) or less and are expressed in the following way:

general case

$$q_{ult}=c\ N_c\ s_c\ d_c\ i_c\ g_c\ b_c + q\ N_q\ s_q\ d_q\ i_q\ g_q\ b_q + 0.5\ B\gamma\ \ N_\gamma\ s_\gamma\ d_\gamma\ i_\gamma\ g_\gamma\ b_\gamma$$

If the ground is purely cohesive ϕ = 0

$$q_{ult} = 5.14\ c\ (1+ s_c +d_c - i_c - g_c- b_c) +q$$

where dc, dq, d γ, are the factors of depth, sc, sq, s γ, are form factors, ic, iq, the γ, are factors load inclination, bc, bq, b γ, are the factors of inclination of laying and gc, gq, g γ, are factors that take into account the fact that the foundation rests on a slope.
The factors Nc, Nq, N γ are expressed as:

$$N_q = e^{\pi tg\ \phi} K_p$$

$$N_c = \left(N_q - 1\right) ctg\phi$$

$$N_\gamma = 1.5\left(N_q - 1\right) tg\phi$$

Now let's see how they express the various factors that appear in the expression of the ultimate load.

Form factors

per ϕ =0

$$s_c = 0.2\frac{B}{L}$$

per ϕ >0

$$s_c = 1+\frac{N_q}{N_c}\frac{B}{L}$$

$$s_q = 1+\frac{B}{L}tg\phi$$

$$s_\gamma = 1-0.4\frac{B}{L}$$

Depth Factors

It defines the parameter k as:

$$k = \frac{D}{B} \quad \text{se} \quad \frac{D}{B} \leq 1$$

$$k = arctg\frac{D}{B} \quad \text{se} \quad \frac{D}{B} \succ 1$$

The various coefficients are expressed as

per $\phi = 0$ $\qquad dc = 0.4\ k$

per $\phi > 0$ $\qquad dc = 1+0.4\ k$

$$d_q = 1 + 2tg\phi\ (1 - sin\phi)^2\ k$$

$$d_\gamma = 1$$

Load inclination Factors

Denote by V H and the load components respectively perpendicular and parallel to the base and Af with the effective area of the foundation obtained as Af = B 'x L' (B 'and L' are related to the actual size of the foundation B, L, and the eccentricity of the load eB, eL by the relations B '= B-2EB L '= L-2EL) η and with the angle of inclination of the foundation expressed in degrees (η = 0 for foundation horizontal).

The factors of inclination of the load is expressed as:

per $\phi = 0$

$$i_c = \frac{1}{2}\left(1 - \sqrt{1 - \frac{H}{A_f c_a}}\right)$$

per $\phi > 0$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1}$$

$$i_q = \left(1 - \frac{0.5H}{V + A_f c_a ctg\phi}\right)^5$$

per $\eta = 0$

$$i_\gamma = \left(1 - \frac{0.7H}{V + A_f c_a ctg\phi}\right)^5$$

per $\eta > 0$

$$i_\gamma = \left(1 - \frac{(0.7 - \eta° / 450°)H}{V + A_f c_a ctg\phi}\right)^5$$

<u>Inclination factors of the laying of the foundation</u>

per $\phi = 0$

$$b_c = \frac{\eta^o}{147^o}$$

per $\phi > 0$

$$b_c = 1 - \frac{\eta^o}{147^o}$$

$$b_q = e^{-2\eta tg\ \phi}$$

$$b_\gamma = e^{-2.7\eta tg\ \phi}$$

<u>Ground slope factors</u>

per $\phi = 0$

$$g_c = \frac{\beta^o}{147^o}$$

per $\phi > 0$

$$g_c = 1 - \frac{\beta^o}{147^o}$$

$$g_q = g_\gamma = (1 - 0.5tg\beta)^5$$

In order to apply the formula of Hansen should be verified the following conditions:

$$H < V \, tg\, \delta + Af\, ca$$
$$\beta \leq \phi$$

$$i_q, i_\gamma > 0$$
$$\beta + \eta \leq 90°$$

3.10.5 VESIC

The formula of Vesic is analogous to the formula of Hansen. Change only the factor N γ and the expression ofsome coefficients.

Reprint entirely all expressions referring to in the previous paragraph any limitations and clarifications.

general case

$$q_{ult}=c\, N_c\, s_c\, d_c\, i_c\, g_c\, b_c + q\, N_q\, s_q\, d_q\, i_q\, g_q\, b_q + 0.5\, B\gamma\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma\, g_\gamma\, b_\gamma$$

If the ground is purely cohesive $\phi = 0$

$$q_{ult} = 5.14\, c\, (1 + s_c + d_c - i_c - g_c - b_c) + q$$

$$N_q = e^{\pi\, tg\, \phi} K_p$$

$$N_c = \left(N_q - 1\right) ctg\phi$$

$$N_\gamma = 2\left(N_q + 1\right) tg\phi$$

Form factors

per ϕ =0 $$s_c = 0.2\frac{B}{L}$$

per ϕ >0 $$s_c = 1 + \frac{N_q}{N_c}\frac{B}{L}$$

$$s_q = 1 + \frac{B}{L} tg\phi$$

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

Depth factors

$$k = \frac{D}{B} \quad \text{se} \quad \frac{D}{B} \leq 1$$

$$k = arctg\frac{D}{B} \quad \text{se} \quad \frac{D}{B} \succ 1$$

The various coefficients are expressed as:

per ϕ = 0 $$d_c = 0.4\ k$$

per ϕ > 0 $$d_c = 1+0.4\ k$$

$$d_q = 1 + 2tg\phi(1 - sin\phi)^2 k$$

$$d_\gamma = 1$$

Inclination load factors

$$m = \frac{2 + B/L}{1 + B/L}$$

per $\phi = 0$

$$i_c = \frac{mH}{A_f c_a N_c}$$

per $\phi > 0$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1}$$

$$i_q = \left(1 - \frac{H}{V + A_f c_a ctg\phi}\right)^m$$

$$i_\gamma = \left(1 - \frac{H}{V + A_f c_a ctg\phi}\right)^{m+1}$$

Inclination of the laying foundation factors

per $\phi = 0$

$$b_c = \frac{\eta^o}{147^o}$$

per $\phi > 0$

$$b_c = 1 - \frac{\eta^o}{147^o}$$

$$b_q = b_\gamma = (1 - \eta tg\phi)^2$$

Inclination soil factors

per $\phi = 0$

$$g_c = \frac{\beta^o}{147^o}$$

per $\phi > 0$

$$g_c = 1 - \frac{\beta^o}{147^o}$$

$$g_q = g_\gamma = (1 - tg\beta)^2$$

3.11 Consideration on the use of bearing capacity formula

The program implements all the four methods described above for the calculation of capacity bearing soil in the foundation.

You choose which formula to adopt the Options window Analysis of the Analysis menu. The ultimate load provided by the various formulas is a unitary ultimate load (force / unit area).

the limit load foundation is then provided by the relation:

$$Q_{\lim} = q_{ult}\ B'L$$
$$con\ \ B' = B - 2e$$

where B and L are the width and the length of the foundation, and e is the eccentricity of the load (Meyerhof). It is therefore clear that the ultimate load, and then the relative safety factor depends, in the case of the retaining wall, other factors being equal also to the entity of the thrust (change in fact the eccentricity).

The formulas of Hansen and Vesic give values of the ultimate load very similar between them. It is however the designer to choose the formula that considers most suitable for personal experience.

Several authors recommend Hansen formula that allows you to take into account all the factors that occur very often in the calculation of a retaining wall (inclined and eccentric load, inclined foundation, foundation in the vicinity of a slope, etc.). By default, the program assumes the Meyerhof method as a method of calculating the limit load.

4. PARAMETRIC ANALYSIS OF EARTH REINFORCED SLOPES

Consider earth reinforced slope shown schematically in Figure 4.1.

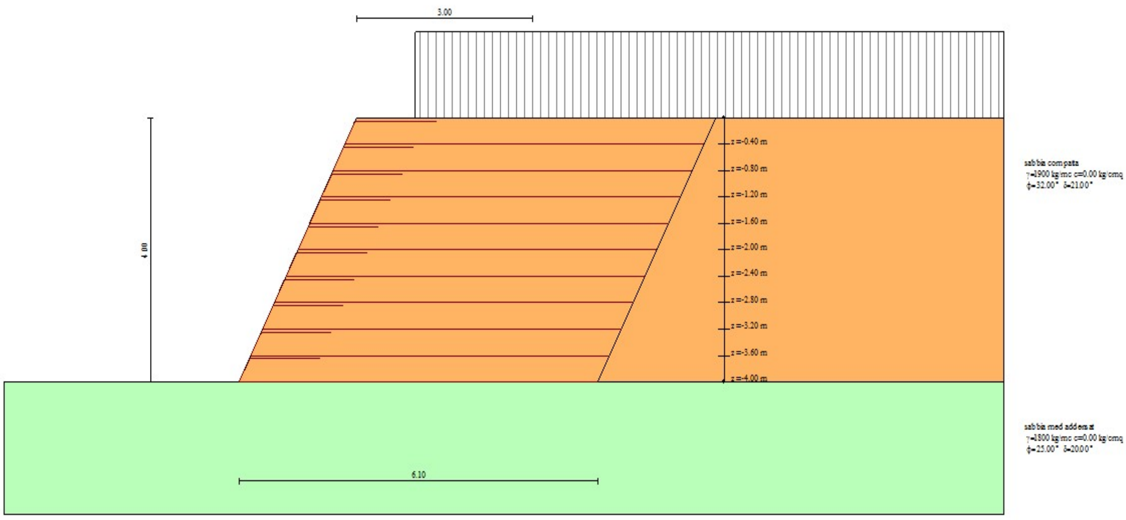


Figure 4.1

The characteristic properties of the foundation soil and the embankment are indicated in the same figure.

The embankment is delimited by a horizontal surface. At the top of the wall acts a vertical uniformly distributed load of intensity characteristic equal to qk. This overload is considered as permanent load.

For permanent actions are considered actions that act during the entire lifetime of system, which increased in intensity over time is so small and slow that it can be considered with sufficient approximation constant in time.

The aquifer is absent or under hydrostatic conditions with free surface placed to the laying of the foundation.

The Ministerial Decree of 14/01/2008 provides that, for retaining walls or mixed structures, should be carried out the checks with reference to at least the following limit states:

• SLU of geotechnical (GEO) and the equilibrium of a rigid body (EQU):

- The overall stability of the complex work of ground-support;
- sliding to the substrate;
- Collapse limit load of the whole foundation-soil;
- Overturning;

• SLU structural (STR):

- Achievement of the resistance in structural elements;

ensuring that the condition Ed <Rd is satisfied for each limit state considered.
The verification of global stability must be carried out according to:

• Approach 1 Combination 2: (A2 + M2 + R2)

taking into account the partial factors given in Table 2.1 and Table 2.2 for the actions and geotechnical parameters and Table 6.8.I (in DM 14/01/2008 in Chapter 6) for safety checks of works of loose materials and excavation fronts.
The remaining tests must be carried at least one of the following two approaches:

• Approach 1: Combination 1: (A1 + M1 + R1)
Combination 2: (A2 + M2 + R2)

• Approach 2: (A1 + M1 + R3)

taking into account the values of the partial factors given in Table 2.1, Table 2.2 and Table 2.3. In the case of retaining walls with anchors to the ground, the checks must be made with reference to only one approach. In tests conducted with the approach 2, which are aimed at structurally sizing, the coefficient R shall not be taken into account.

The state tipping does not provide for the mobilization of soil strength foundation and should be treated as a state of equilibrium limit as a rigid body (EQU), using the partial factors on actions in Table 2.1 and using partial factors of group (M2) for the calculation of forces.

4.1 Design approach 1 [DA1]

With this approach it is necessary to carry out all checks to the ultimate limit state with both set combinations.

4.1.1 Combination 1 [DA1.C1]

With this a combination increases permanent and variable actions with appropriate partial safety factors, but does not change the resistance of the ground on the back of the wall and the foundation soil (unit are partial factors on soil parameters). Are also unitary the coefficient γE γR on the effects of actions and resistances overall system.

Table 4.1 shows the values of the active thrust

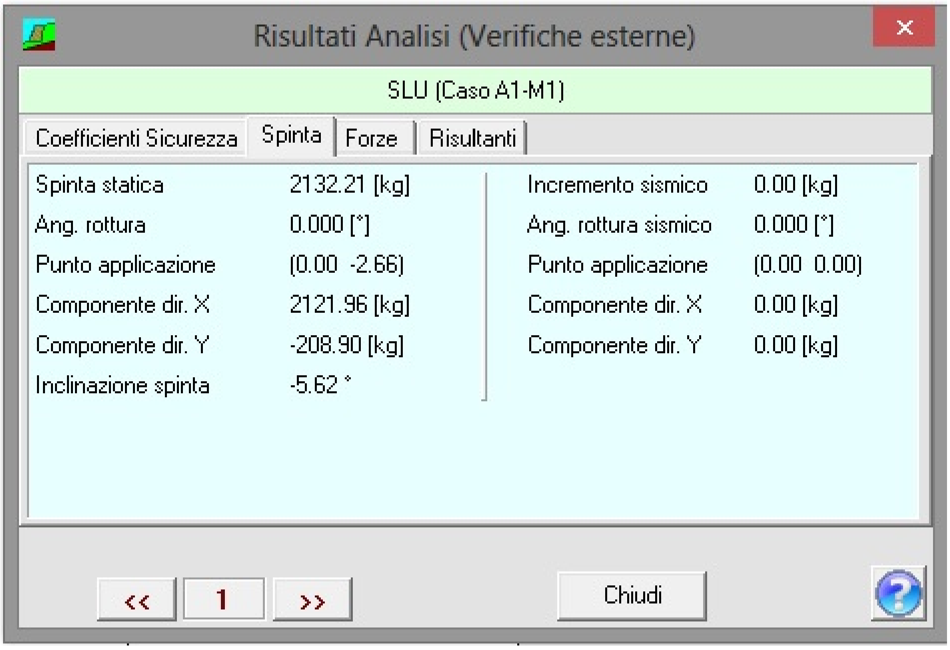
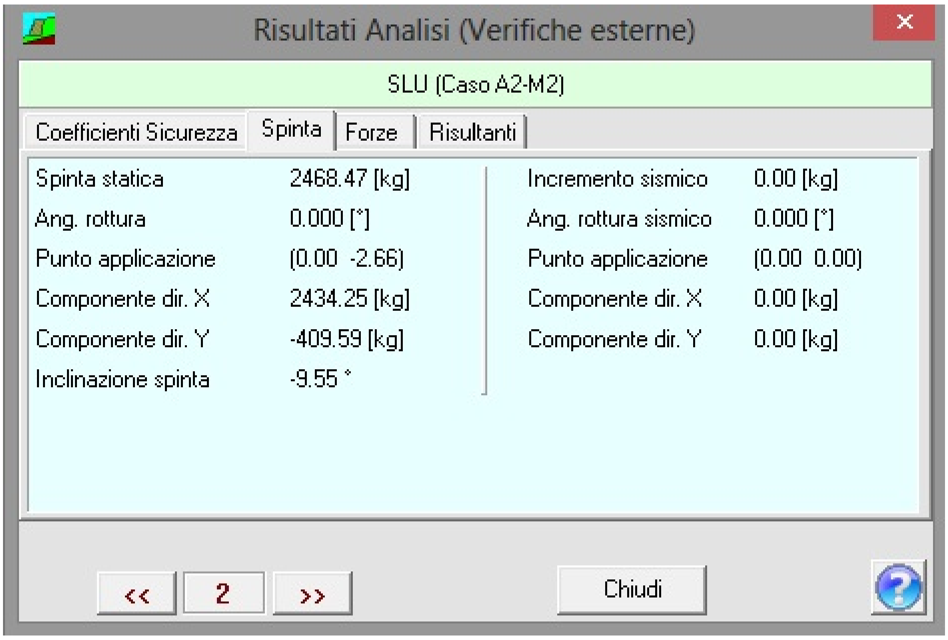


Table 4.1

4.1.2 Combination 2 [DA1.C2]

With this combination increased the only variable actions with appropriate partial safety factors, and change the resistance of the ground on the back of the wall and those of the foundation soil. Table 4.2 shows the values of the active thrust



Risultati Analisi (Verifiche esterne)

SLU (Caso A2-M2)

Coefficienti Sicurezza | Spinta | Forze | Risultanti

Spinta statica	2468.47 [kg]	Incremento sismico	0.00 [kg]
Ang. rottura	0.000 [°]	Ang. rottura sismico	0.000 [°]
Punto applicazione	(0.00 -2.66)	Punto applicazione	(0.00 0.00)
Componente dir. X	2434.25 [kg]	Componente dir. X	0.00 [kg]
Componente dir. Y	-409.59 [kg]	Componente dir. Y	0.00 [kg]
Inclinazione spinta	-9.55 °		

<< 2 >> Chiudi

Table 4.2

4.2 Parametric analysis

The parametric study has the aim to provide a quantitative comparison between the results obtained with the new DM 14/01/2008 by varying the friction angle of the foundation soil and the seismic acceleration device of the site.

Figure 4.2 shows the scheme of the reinforced soil taken into consideration. The dimensions of the section are expressed as a function of the height of the wall H and base B. The surface of the embankment has been regarded as horizontal.

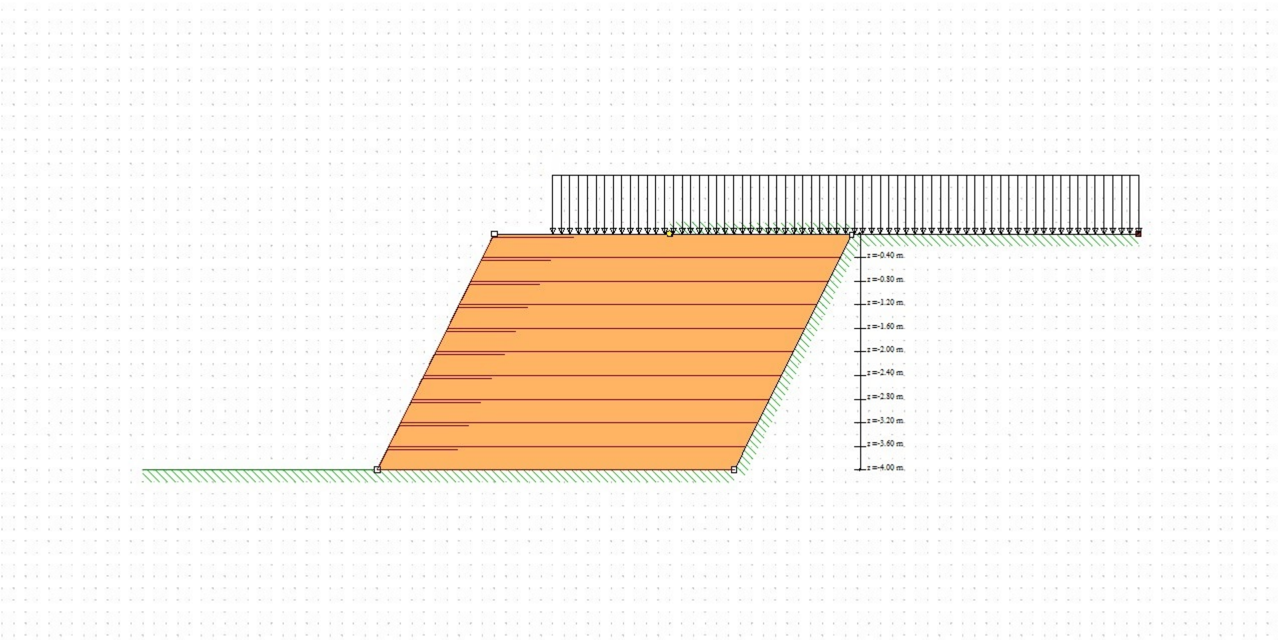


Figure 4.2

On the embankment acts a uniformly distributed load with intensity equal to qk. For the soil has been considered a unit weight of 19 kN/m3, while for the Foundation soil it is assumed 18 kN/m3.

The study was conducted by searching for each design approach provided by DM 14/01/2008, the minimum base width usingthe ratio B / H, that allows to simultaneously meet both the ultimate limit states previously described.

The results of the parametric study is summarized in the following graphs (figure 4.3,4.4,4.5) that provide the value of the ratio B / H as a function of the mechanical characteristics of the embankment (angle of friction characteristic Φterr) and of the foundation soil (angle of friction characteristic Φfond) varying the seismic acceleration of the site from 0.06 to 0.25.

Φ (friction angle)	ag	B/H
25	0.06	1.525
	0.08	1.5375
	0.15	1.75
	0.2	1.85
	0.25	1.975

Table 4.3

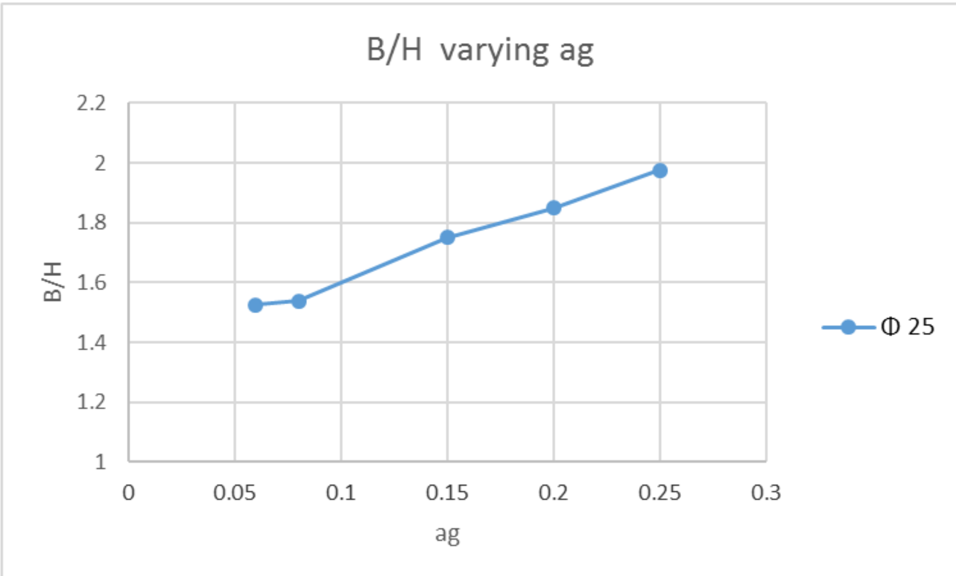


Figure 4.3

Φ (friction angle)	ag	B/H
30	0.06	1.05
	0.08	1.075
	0.15	1.175
	0.2	1.2
	0.25	1.25

Table 4.4

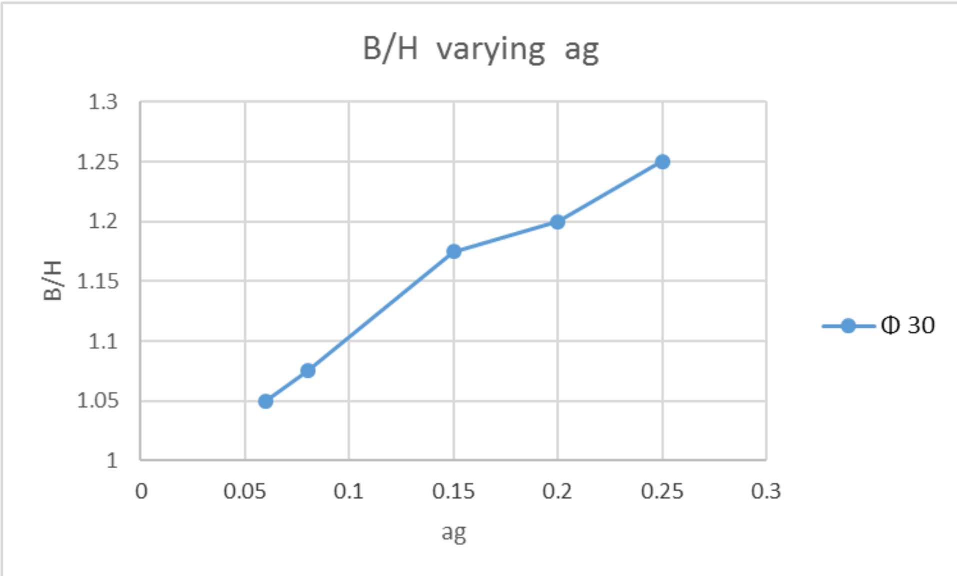


Figure 4.4

Φ (friction angle)	ag	B/H
35	0.06	0.8
	0.08	0.8125
	0.15	0.875
	0.2	0.9
	0.25	0.925

Table 4.5

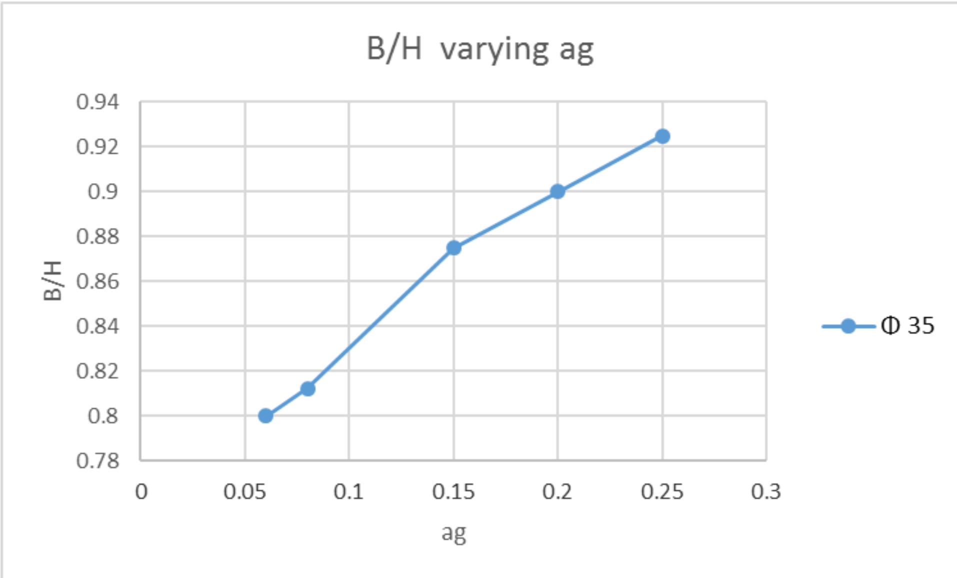


Figure 4.5

it should be noted as the ratio B / H varies significantly having a foundation soil with low Φ while coming to soils with high friction angle (better for building) the report gives a little increase.

4.3 Comparison between [DA1.C1] and [DA1.C2]

In the figure 4.6 we compare the safety factor for sliding in the two approaches. (DA1.C1, DA1.C2)

Φ (friction angle)	SLU (A1 M1)	SLU (A2 M2)
	sliding	sliding
25	10.24	5.21
	10.31	5.5
	11.83	6.32
	12.51	6.68
	13.35	7.13

Table 4.6

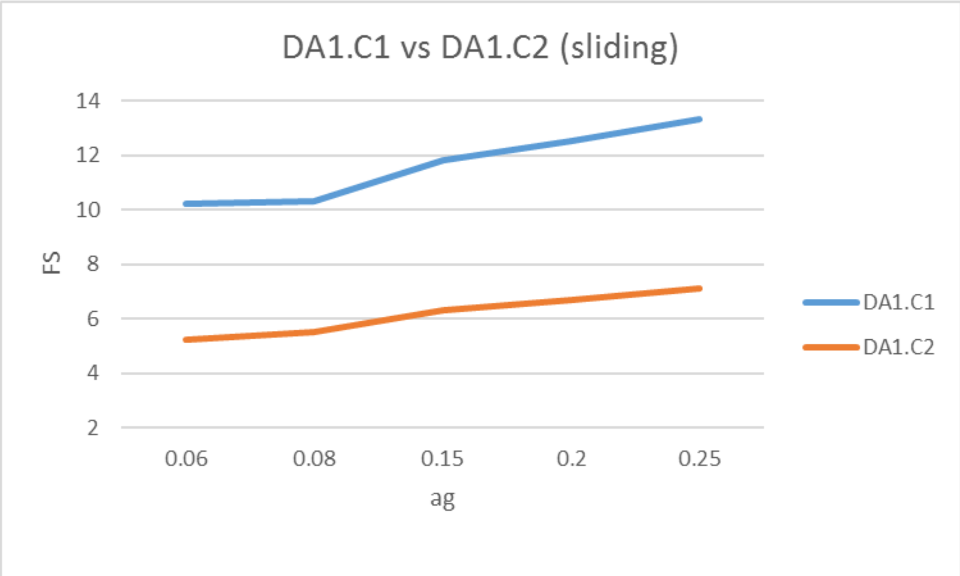


Figure 4.6

From this graph we can see how the DA1.C2 are more precautionary

In the figure 4.7 we compare the safety factor for limit load in the two approaches.

Φ (friction angle)	SLU (A1 M1)	SLU (A2 M2)
	limit load	limit load
25	2.11	1.1
	2.2	1.15
	2.72	1.45
	2.95	1.58
	3.25	1.75

Table 4.7

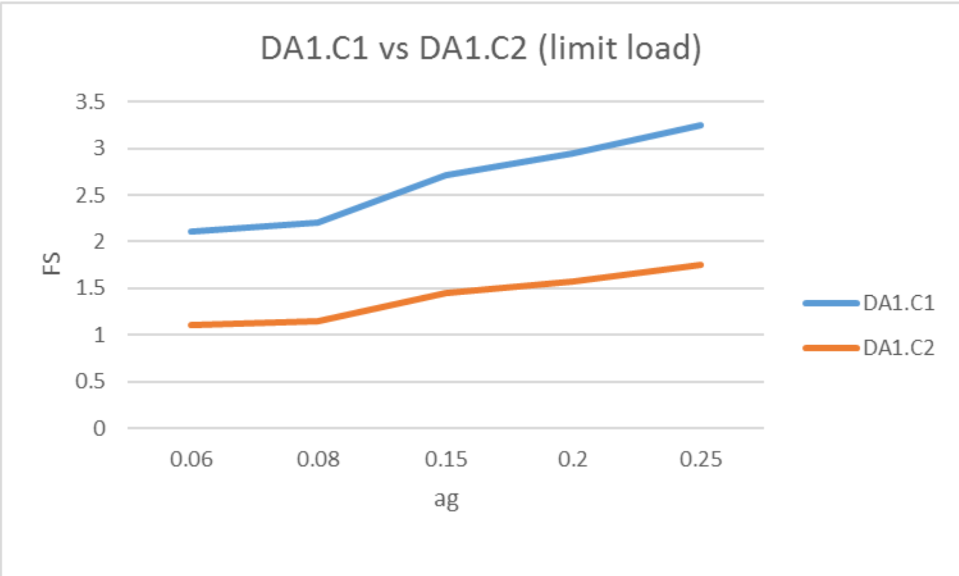


Figure 4.7

From this graph we can see how the DA1.C2 are more precautionary

4.3.1 [DA1.C1] Varying friction angle of the soil foundation

In figure 4.8 and 4.9 we compare the safety factor for sliding with DA1.C1 approaches varying the friction angle of the foundation soil.

Φ (friction angle)	ag	SLU (A1 M1)
		sliding
25	0.06	10.24
	0.08	10.31
	0.15	11.83
	0.2	12.51
	0.25	13.35
30	0.06	7.09
	0.08	7.26
	0.15	7.94
	0.2	8.11
	0.25	8.45
35	0.06	5.4
	0.08	5.4
	0.15	5.91
	0.2	6.08
	0.25	6.25

Table 4.8

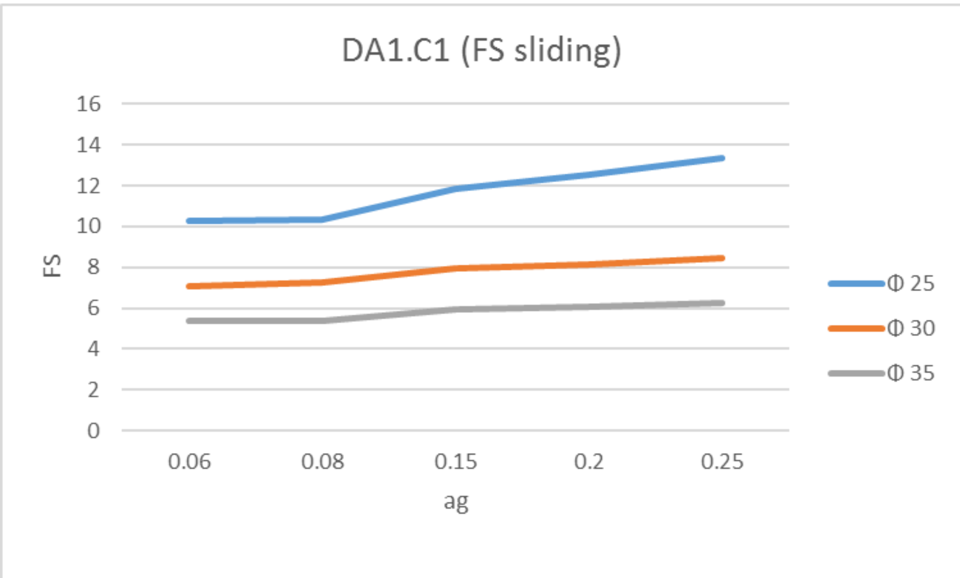


Figure 4.8

Φ (friction angle)	ag	SLU (A1 M1)
		limit load
25	0.06	2.11
	0.08	2.2
	0.15	2.72
	0.2	2.95
	0.25	3.25
30	0.06	2.75
	0.08	2.87
	0.15	3.338
	0.2	3.5
	0.25	3.77
35	0.06	3.8
	0.08	3.8
	0.15	4.64
	0.2	4.93
	0.25	5.22

Table 4.9

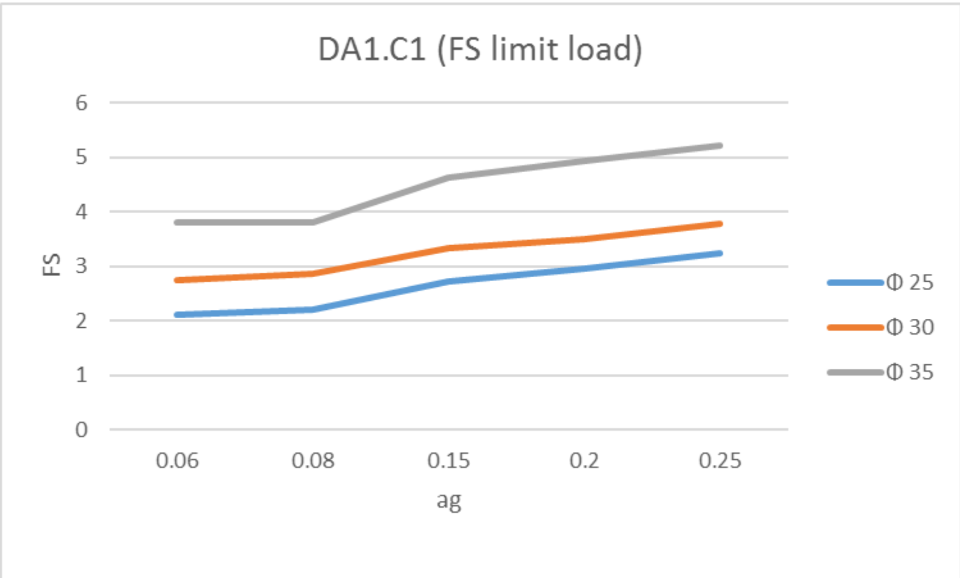


Figure 4.9

For the sliding condition we see that increasing the friction angle of the foundation, for equal seismic acceleration, the security factor decrease.

On the other hand for the limit load condition when we increase the friction angle, the security factor increase.

4.3.2 [DA1.C1] Related to a negative earthquake

now let's see how the structure behaves in the event of a negative earthquake with tests to sliding figure 4.10 and to limit load figure 4.11.

Φ (friction angle)	ag	negative e.quake	
		sliding	limit load
25	0.06	7.19	2.6
	0.08	6.77	2.67
	0.15	4.25	2.83
	0.2	3.78	2.9
	0.25	3.29	2.94
30	0.06	5.44	3.45
	0.08	5.24	3.58
	0.15	3.51	3.79
	0.2	3.13	3.77
	0.25	2.76	3.8
35	0.06	4.34	4.89
	0.08	4.14	4.88
	0.15	2.96	5.65
	0.2	2.7	5.83
	0.25	2.39	5.92

Table 4.10

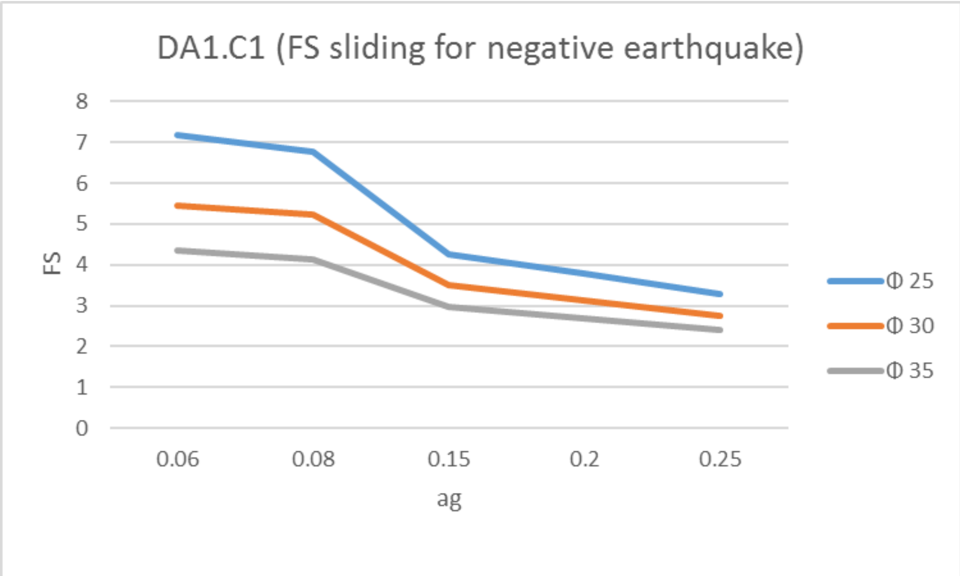


Figure 4.10

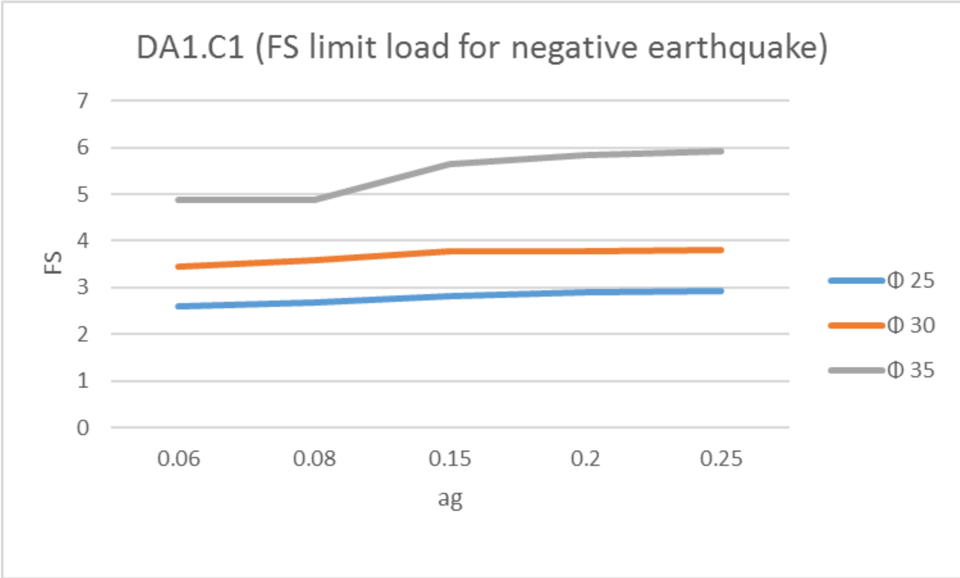


Figure 4.11

For the sliding condition with negative earthquake we see that increasing the friction angle of the foundation, for equal seismic acceleration, the security factor decrease.

On the other hand for the limit load condition with negative earthquake when we increase the friction angle, the security factor increase.

4.3.3 [DA1.C1] Related to a positive earthquake

And now the same tests were made with a positive earthquake, figure 4.12 and 4.13.

Φ (friction angle)	ag	positive e.quake	
		sliding	limit load
25	0.06	7.21	2.57
	0.08	6.81	2.63
	0.15	4.38	2.73
	0.2	3.94	2.79
	0.25	3.47	2.82
30	0.06	5.45	3.41
	0.08	5.26	3.53
	0.15	3.6	3.65
	0.2	3.25	3.61
	0.25	2.9	3.63
35	0.06	4.35	4.83
	0.08	4.15	4.81
	0.15	3.04	5.43
	0.2	2.78	5.57
	0.25	2.5	5.62

Table 4.11

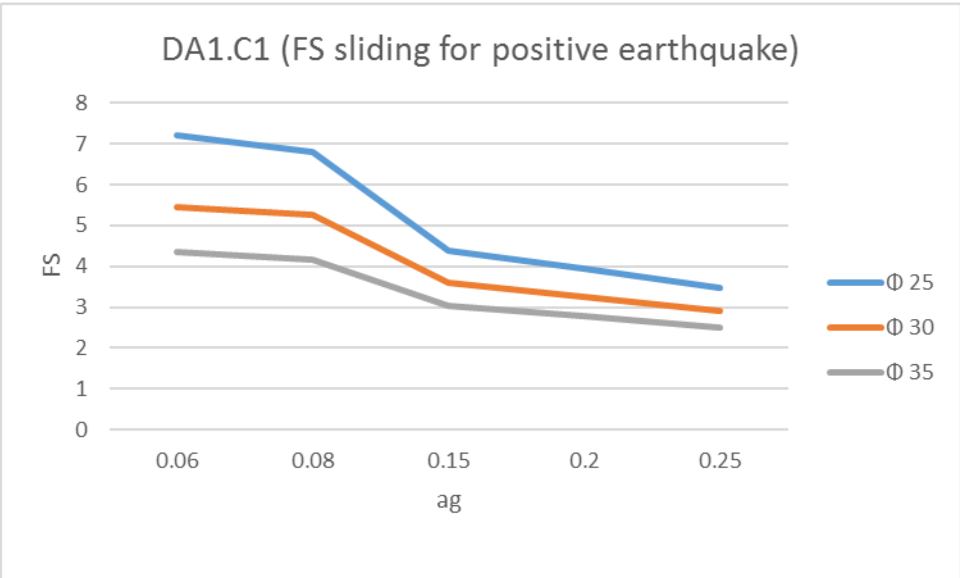


Figure 4.12

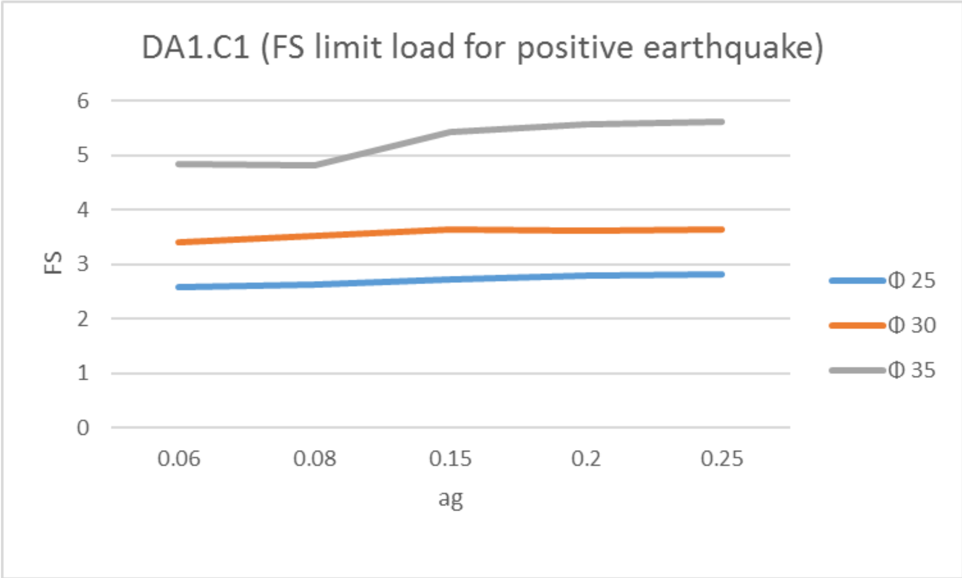


Figure 4.13

For the sliding condition with positive earthquake we see that increasing the friction angle of the foundation, for equal seismic acceleration, the security factor decrease.

On the other hand for the limit load condition with negative earthquake when we increase the friction angle, the security factor increase.

4.4 [DA1.C2] Varying friction angle of the soil foundation

Now we analyze the same system through the approach DA1.C2

In figure 4.14 and 4.15 we compare the safety factor for sliding with DA1.C2 approaches varying the friction angle of the foundation soil.

Φ (friction angle)	ag	SLU (A2 M2)	
		sliding	limit load
25	0.06	5.21	1.1
	0.08	5.5	1.15
	0.15	6.32	1.45
	0.2	6.68	1.58
	0.25	7.13	1.75
30	0.06	3.77	1.3
	0.08	3.86	1.36
	0.15	4.23	1.62
	0.2	4.32	1.68
	0.25	4.5	1.8
35	0.06	2.87	1.65
	0.08	2.87	1.65
	0.15	3.14	2.02
	0.2	3.23	2.14
	0.25	3.32	2.27

Table 4.12

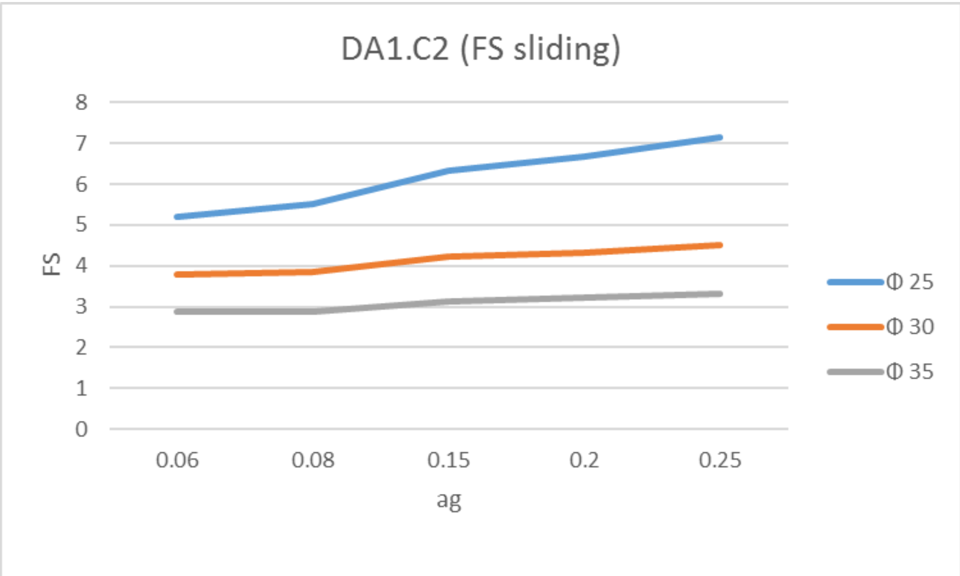


Figure 4.14

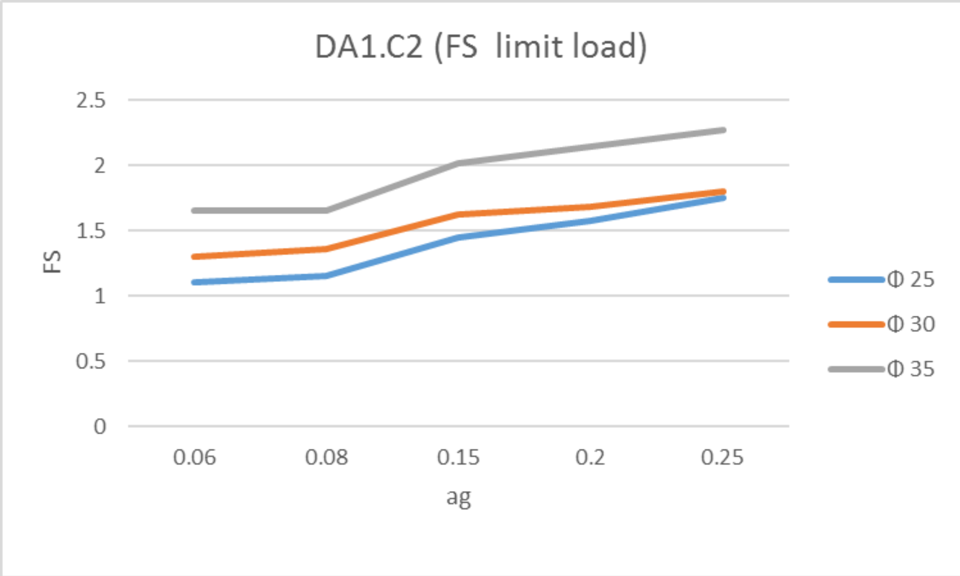


Figure 4.15

For the sliding condition we see that increasing the friction angle of the foundation, for equal seismic acceleration, the security factor decrease.

On the other hand for the limit load condition when we increase the friction angle, the security factor increase.

4.4.1 [DA1.C2] Related to a negative earthquake

let's see how the structure behaves in the event of a negative earthquake with tests to sliding figure 4.16 and to limit load figure 4.17.

Φ (friction angle)	ag	negative e.quake	
		sliding	limit load
25	0.06	4.05	1.01
	0.08	4.04	1.04
	0.15	2.83	1.04
	0.2	2.57	1.04
	0.25	2.29	1.02
30	0.06	3.11	1.22
	0.08	3.04	1.26
	0.15	2.24	1.26
	0.2	2.04	1.22
	0.25	1.84	1.19
35	0.06	2.44	1.6
	0.08	2.36	1.59
	0.15	1.84	1.72
	0.2	1.71	1.73
	0.25	1.55	1.7

Table 4.13

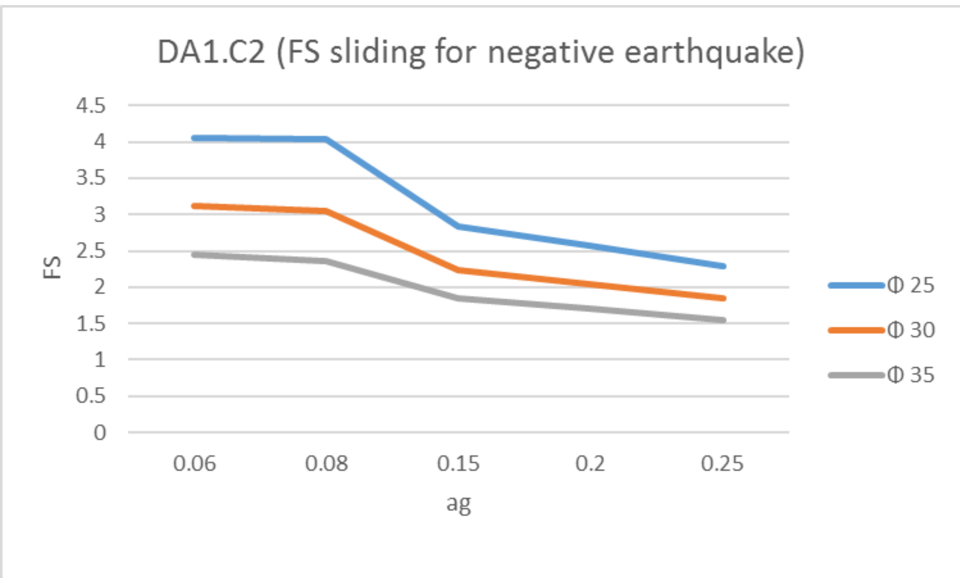


Figure 4.16

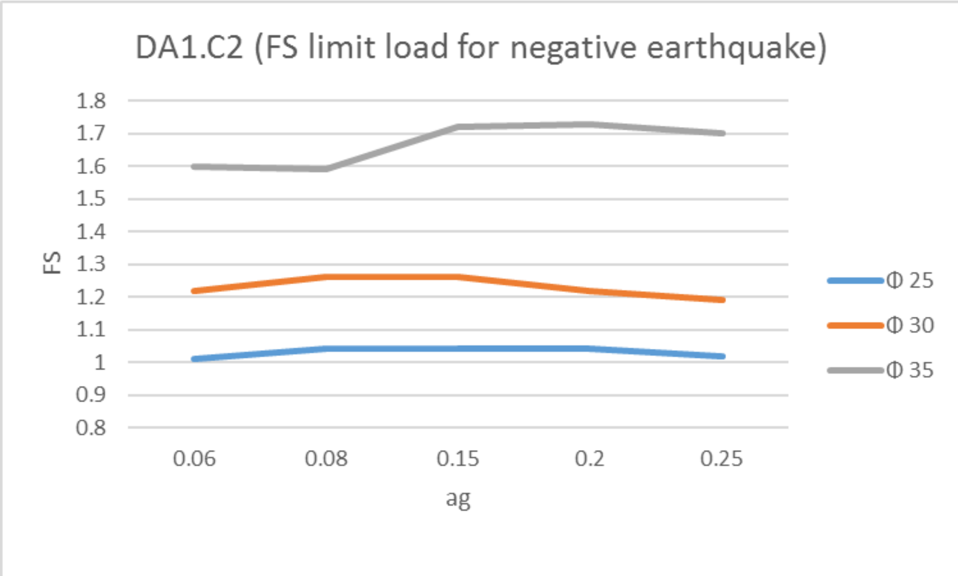


Figure 4.17

For the sliding condition with negative earthquake we see that increasing the friction angle of the foundation, for equal seismic acceleration, the security factor decrease.

On the other hand for the limit load condition with negative earthquake when we increase the friction angle, the security factor increase.

the problem here occurs when Φ for the foundation is equal to 25. in this case the structure will be optimized looking only DA1.C2 values relatively to the case corresponding to the limit load in seismic condition.

4.4.2 [DA1.C2] Related to a positive earthquake

Tests were made with a positive earthquake, figure 4.18 and 4.19.

Φ (friction angle)	ag	positive e.quake	
		sliding	limit load
25	0.06	4.07	1
	0.08	4.06	1.02
	0.15	2.9	1.01
	0.2	2.67	1.01
	0.25	2.4	1
30	0.06	3.12	1.21
	0.08	3.05	1.25
	0.15	2.29	1.22
	0.2	2.11	1.18
	0.25	1.92	1.16
35	0.06	2.45	1.58
	0.08	2.36	1.57
	0.15	1.88	1.66
	0.2	1.76	1.67
	0.25	1.61	1.64

Table 4.14

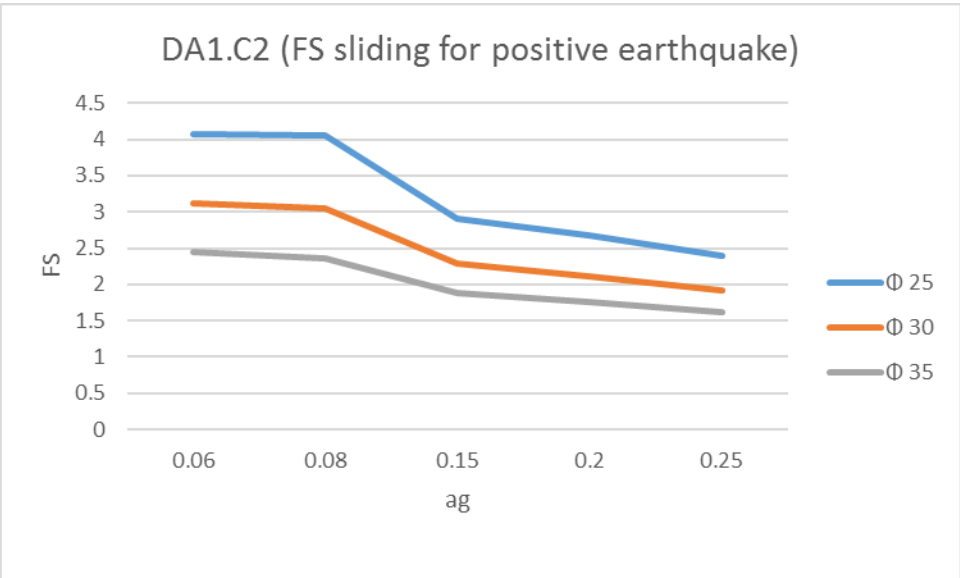


Figure 4.18

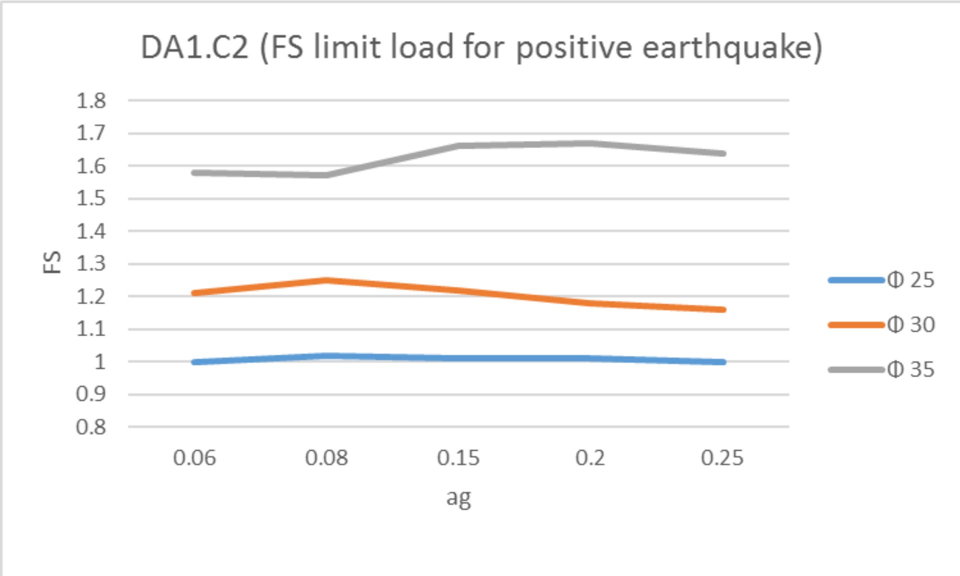


Figure 4.19

For the sliding condition with positive earthquake we see that increasing the friction angle of the foundation, for equal seismic acceleration, the security factor decrease.
On the other hand for the limit load condition with negative earthquake when we increase the friction angle, the security factor increase.

Here we have the same conditions that we see for the negative earthquake and for the same reasons we work for optimize the structure only with DA1.C2 for limit load in seismic condition.

4.5 Global stability

now let's see how the structure behaves toward the global stability changing the foundation soil and seismic acceleration.

Φ (friction angle)	ag	global stability	
		positive e.quake	negative e.quake
25	0.06	1.59	1.59
	0.08	1.13	1.13
	0.15	1.15	1.15
	0.2	1.16	1.17
	0.25	1.16	1.17
30	0.06	1.1	1.1
	0.08	1.11	1.11
	0.15	1.1	1.11
	0.2	1.1	1.1
	0.25	1.1	1.11
35	0.06	1.11	1.11
	0.08	1.1	1.1
	0.15	1.11	1.11
	0.2	1.11	1.11
	0.25	1.1	1.11

Table 4.15

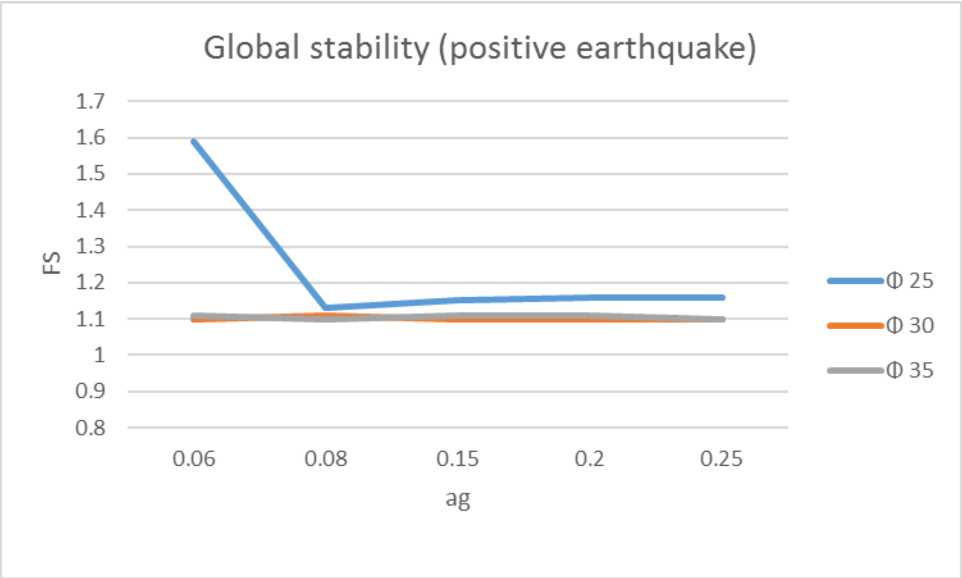


Figure 4.20

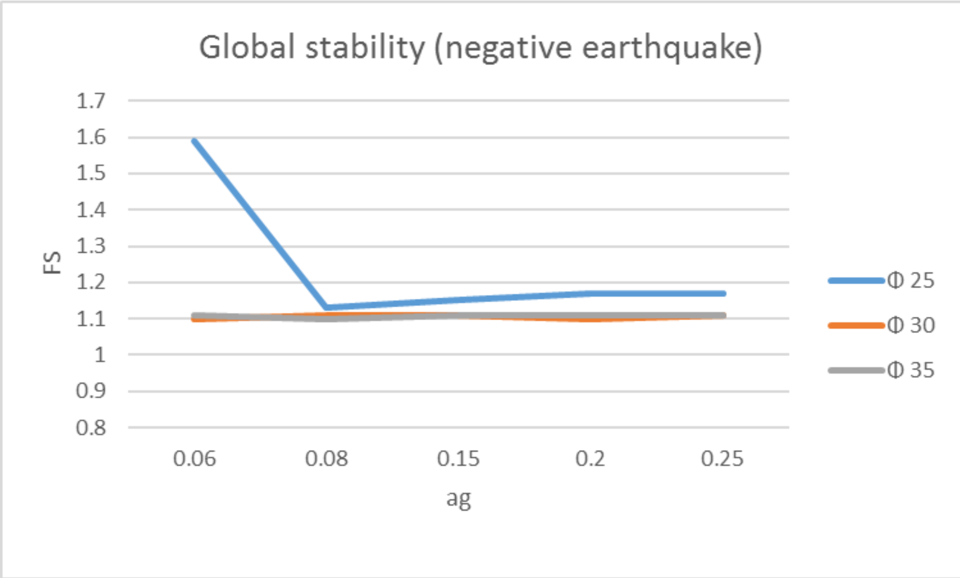


Figure 4.21

The problems for foundation soils with Φ equal to 30 and 35 come when you switch to the global stability.

In this case the designer will not have to watch the verifications to limit load for the DA1.C2 but will have to focus only on the checks to overall stability in the presence of earthquake.

The examination of the tables and graphs can lead to the following conclusions:

• The limit state varies depending on the friction angle of the soil, if for Φ equal to 25 the constraint was the limit load in seismic conditions, for Φ equal to 30 and 35 of the constraint is the global stability.

• Relatively to the approach 1 the combination C2 is always more cautionary of C1.

5. TECHNICAL RELATION

Take into account earth reinforced slope shown schematically in Figure 5.1

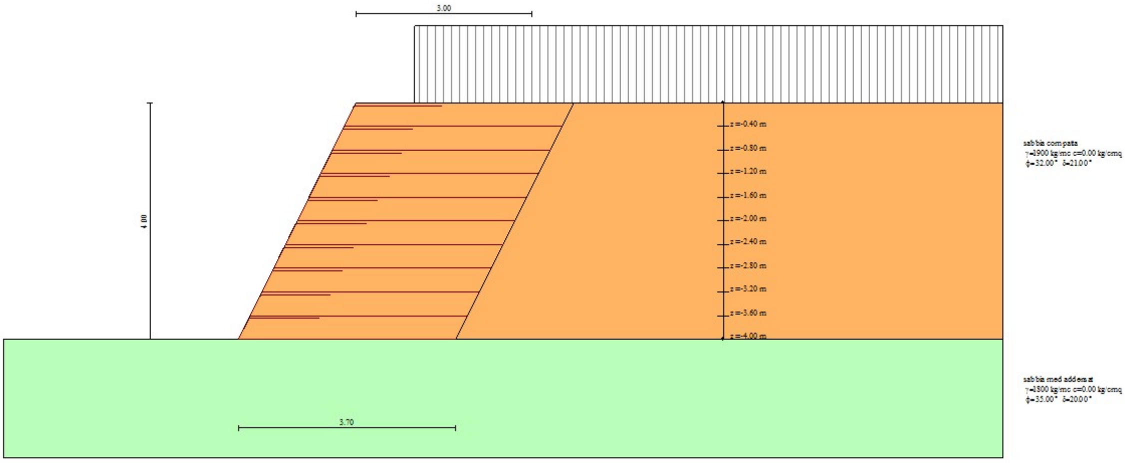


Figure 5.1

5.1 Soils Description

symbols used

Description terrain

γ unit weight of the soil expressed in [kg / m³]

γ sat saturated unit weight of the soil expressed in [kg / m³]

ϕ angle of internal friction of the soil, expressed in degrees

δ pile-soil friction angle in degrees

c cohesion of the ground expressed in [kg / cm]

ca Adhesion of the ground expressed in [kg / cm]

Descrizione	γ	γ_{sat}	ϕ	δ	c	ca
compact sand	1900.00	2000.00	32.00	21.00	0.000	0.000
thickened sand	1800.00	2000.00	35.00	20.00	0.000	0.000

Ground pushing stratigraphy

symbols used

SP - layer thickness, expressed in [m]

Inc - inclination of the layer, expressed in [°]

Soil - earth of the layer

N	Sp	Inc	Soil
1	4.00	0.00	compact sand
2	2.00	0.00	thickened sand

Soil profile
symbols used

X Y - coordinate of this point, expressed in [m]
Y - Intercept point, expressed in [m]

n°	X	Y
1	8.00	0.00

5.2 Reinforcements characteristics
symbols used

Reinforcement ID of the reinforcement
LTDS Resistance of long-term project, expressed in [kg / m]
FSDG Safety factor for joint damage
FSDC Safety factor for chemical damage
FSDB Safety factor for biological damage
FSDA safety factor for environmental damage
LTDSA resistance allowable long-term project, expressed in [kg / m]

Reinforce	LTDS	FS_{DG}	FS_{DC}	FS_{DB}	FS_{DA}	$LTDS_{A}$
reinforce 1	2000.00	1.00	1.00	1.00	1.30	1153.85

Reinforced hearth geometry

symbols used

The reference system is the point at the top right of reinforced earth
Abscissa X, expressed in [m]
Ordinate Y, expressed in [m]

n°	X	Y
1	0.00	0.00
2	-3.00	0.00
3	-5.00	-4.00
4	-1.30	-4.00
5	0.70	0.00

Description of reinforcements
symbols used

z height of the reinforcement
L The length of the reinforcement, expressed in [m]
LRV length stretch of vertical turn (front lapel), expressed in [m]
LRO length stretch of horizontal flap (inside of the flap), expressed in [m]

z	Reinforce	L	Lrv	Lro
-0.40	reinforce 1	3.70	0.35	1.50

-0.80	reinforce 1	3.70	0.35	1.20
-1.20	reinforce 1	3.70	0.35	1.20
-1.60	reinforce 1	3.70	0.35	1.20
-2.00	reinforce 1	3.70	0.35	1.20
-2.40	reinforce 1	3.70	0.35	1.20
-2.80	reinforce 1	3.70	0.35	1.20
-3.20	reinforce 1	3.70	0.35	1.20
-3.60	reinforce 1	3.70	0.35	1.20
-4.00	reinforce 1	3.70	0.35	1.20

5.3 Load conditions

Symbols and sign conventions adopted
Positive vertical loads down.
Positive horizontal loads to the left.
Positive moment counterclockwise.
X coordinate of point of application of the concentrated load expressed in [m]
Fx Horizontal component of the concentrated load expressed in [kg]
Fy vertical component of the concentrated load expressed in [kg]
Xi starting point of the distributed load expressed in [m]
Xf end point of the distributed load expressed in [m]
Qi intensity of the load at x = Xi expressed in [kg / m]
Qf intensity of the load at x = Xf expressed in [kg / m]
D / C type load: D = distribuited C = concentrated
Ψ 0, Ψ 1, Ψ 2 combination coefficients

Condition n° 1 - PERMANENT - (Condition 1)

Distribuited load

X_i	X_f	Q_i	Q_f
-2.00	8.00	2000	2000

Calculation options

The checks bearing capacity were performed by the method of MEYERHOF.
The global stability checks and compound were performed by the method of FELLENIUS.

Load combinations description

symbols used

γ coefficient of participation of the condition
Ψ Coefficient combination of the condition
C Coefficient of total participation of the condition

Combination n° 1 SLU (Case A1-M1)

Condition	**γ**	**Ψ**	**C**	**Effect**
weight	1.30	1.00	1.30	Unfavorable
Soil push	1.30	1.00	1.30	Unfavorable
Condition 1	1.30	1.00	1.30	Unfavorable

Combination n° 2 SLU (Case A2-M2)

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 3 EQU

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	0.90	1.00	0.90	Favorable
Soil push	1.10	1.00	1.10	Unfavorable
Condition 1	1.10	1.00	1.10	Unfavorable

Combination n° 4 STAB

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 5 SLU (Case A1-M1) - negative e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 6 SLU (Case A1-M1) - positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 7 SLU (Case A2-M2) - positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 8 SLU (Case A2-M2) – negativo e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 9 EQU - Negative e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Favorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 10 EQU -. Positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Favorevole
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 11 STAB - Positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condizione 1	1.00	1.00	1.00	Unfavorable

Combination n° 12 STAB - Negative e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 13 SLE (Almost Permanent)

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 14 SLE (Frequent)

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 15 SLE (Rare)

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 16 SLE (Almost Permanent) - positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable

Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 17 SLE (almost Permanent) - negative e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 18 SLE (Frequent) - positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 19 SLE (Frequent) - Negative e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 20 SLE (Rare) - Positive e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

Combination n° 21 SLE (Rare) - Negative e.quake

Condizione	**γ**	**Ψ**	**C**	**Effetto**
weight	1.00	1.00	1.00	Unfavorable
Soil push	1.00	1.00	1.00	Unfavorable
Condition 1	1.00	1.00	1.00	Unfavorable

5.4 Checks - Safety factors

symbols used

FSRib overturning safety factor
FSScor slide Safety Factor
FSQlim limit load safety factor
FSStab safety factor to global stability

	FS_{Rib}	FS_{Scor}	FS_{Qlim}	FS_{Stab}
Comb. n° 1 SLU (Case A1-M1)	--	6.25	5.22	--
Comb. n° 2 SLU (Case A2-M2)	--	3.32	2.27	--
Comb. n° 3 EQU	12.38	--	--	--
Comb. n° 4 STAB	--	--	--	1.23
Comb. n° 5 SLU (Case A1-M1) – Vert. negative e.quake	--	2.39	5.92	--
Comb. n° 6 SLU (Case A1-M1) - Vert. positive e.quake	--	2.50	5.62	--
Comb. n° 7 SLU (Case A2-M2) - Vert. positive e.quake	--	1.61	1.64	--
Comb. n° 8 SLU (Case A2-M2) - Vert. negative e.quake	--	1.55	1.70	--

Comb. n° 9 EQU - Vert. negative e.quake	5.69	--	--	--
Comb. n° 10 EQU - Vert. positive e.quake	7.04	--	--	--
Comb. n° 11 STAB - Vert. positive e.quake	--	--	--	1.10
Comb. n° 12 STAB - Vert. negative e.quake	--	--	--	1.11
Comb. n° 13 SLE (Almost Permanent)	--	6.25	6.78	--
Comb. n° 14 SLE (Frequent)	--	6.25	6.78	--
Comb. n° 15 SLE (Rare)	--	6.25	6.78	--
Comb. n° 16 SLE (Almost Permanent) - Vert. positive e.quake	--	4.67	6.55	--
Comb. n° 17 SLE (Almost Permanent) - Vert. negative e.quake	--	4.65	6.64	--
Comb. n° 18 SLE (Frequent) - Vert. positive e.quake	--	4.67	6.55	--
Comb. n° 19 SLE (Frequent) - Vert. negative e.quake	--	4.65	6.64	--
Comb. n° 20 SLE (Rare) - Vert. positive e.quake	--	4.67	6.55	--
Comb. n° 21 SLE (Rare) - Vert. negative e.quake	--	4.65	6.64	--

Internal checks

symbols used

FSScor Sliding Safety Factor
FSSfil Pull-out Safety factor
FSTraz Tensile Safety Factor
FSScorr Safety factor of sliding flap

	FS_{Scor}	FS_{Sfil}	FS_{Traz}	FS_{ScorR}
Comb. n° 1 SLU (Case A1-M1)	24.73	49.47	2.30	3.12
Comb. n° 2 SLU (Case A2-M2)	11.73	23.47	2.00	1.67
Comb. n° 5 SLU (Case A1-M1) – Vert. negative e.quake	18.43	36.85	2.40	2.52
Comb. n° 6 SLU (Case A1-M1) - Vert. positive e.quake	17.83	35.66	2.27	2.41
Comb. n° 7 SLU (Case A2-M2) - Vert. positive e.quake	8.25	16.51	1.58	1.34
Comb. n° 8 SLU (Case A2-M2) - Vert. negative e.quake	8.48	16.95	1.67	1.40
Comb. n° 13 SLE (Almost Permanent)	24.73	49.47	2.98	3.12
Comb. n° 14 SLE (Frequent)	24.73	49.47	2.98	3.12
Comb. n° 15 SLE (Rare)	24.73	49.47	2.98	3.12
Comb. n° 16 SLE (Almost Permanent) - Vert. positive e.quake	22.90	45.80	2.79	2.84
Comb. n° 17 SLE (Almost Permanent) - Vert. negative e.quake	23.16	46.32	2.83	2.87
Comb. n° 18 SLE (Frequent) - Vert. positive e.quake	22.90	45.80	2.79	2.84
Comb. n° 19 SLE (Frequent) - Vert. negative e.quake	23.16	46.32	2.83	2.87
Comb. n° 20 SLE (Rare) - Vert. positive e.quake	22.90	45.80	2.79	2.84
Comb. n° 21 SLE (Rare) - Vert. negative e.quake	23.16	46.32	2.83	2.87

Composed checks

symbols used

FSComp Safety factor for local stability (compound)

	FS_{Comp}
Comb. n° 4 STAB	1.40
Comb. n° 11 STAB - Vert. Positive e.quake	1.36
Comb. n° 12 STAB - Vert. negative e.quake	1.31

5.5 Esternal checks

Push results

Combination n° 1 SLU (Case A1-M1)

Static push	2132.22	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2121.96	[kg]
Vertical component of static thrust	-208.90	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Combination n° 2 SLU (Case A2-M2)

Static push	2468.47	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2434.26	[kg]
Vertical component of static thrust	-409.59	[kg]
Inclination of thrust compare to the horizontal line	-9.55	[°]

Combination n° 3 EQU

Static push	2715.32	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2677.68	[kg]
Vertical component of static thrust	-450.55	[kg]
Inclination of thrust compare to the horizontal line	-9.55	[°]

Combination n° 5 SLU (Case A1-M1) - Vert. negative e.quake

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	444.11	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	441.98	[kg]
Vertical component of seismic increase	-43.51	[kg]

Combination n° 6 SLU (Case A1-M1) - Vert. positive e.quake

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	558.57	[kg]
Inclination of break line in seismic condition	0.00	[°]

Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	555.89	[kg]
Vertical component of seismic increase -54.73	[kg]	

<u>Combination n° 7 SLU (Case A2-M2) - Vert. positive e.quake</u>

Static push	2468.47	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2434.26	[kg]
Vertical component of static thrust	-409.59	[kg]
Inclination of thrust compare to the horizontal line	-9.55	[°]
Seismic increase of thrust	728.84	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	718.74	[kg]
Vertical component of seismic increase	-120.94	[kg]

<u>Combination n° 8 SLU (Case A2-M2) - Vert. Negative e.quake</u>

Static push	2468.47	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2434.26	[kg]
Vertical component of static thrust	-409.59	[kg]
Inclination of thrust compare to the horizontal line	-9.55	[°]
Seismic increase of thrust	555.67	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	547.97	[kg]
Vertical component of seismic increase	-92.20	[kg]

<u>Combination n° 9 EQU - Vert. Negative e.quake</u>

Static push	2468.47	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2434.26	[kg]
Vertical component of static thrust	-409.59	[kg]
Inclination of thrust compare to the horizontal line	-9.55	[°]
Seismic increase of thrust	555.67	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	547.97	[kg]
Vertical component of seismic increase	-92.20	[kg]

<u>Combination n° 10 EQU - Vert. Positive e.quake</u>

Static push	2468.47	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	2434.26	[kg]

Vertical component of static thrust	-409.59	[kg]
Inclination of thrust compare to the horizontal line	-9.55	[°]

Seismic increase of thrust	728.84	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	718.74	[kg]
Vertical component of seismic increase	-120.94	[kg]

Combination n° 13 SLE (Almost Permanent)

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Combination n° 14 SLE (Frequent)

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Combination n° 15 SLE (Rare)

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Combination n° 16 SLE (Almost Permanent) -Vert. positive e.quake

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	116.57	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	116.01	[kg]
Vertical component of seismic increase	-11.42	[kg]

Combination n° 17 SLE (Almost Permanent) - Vert. Negative e.quake

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	

Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	90.36	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust spinta	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	89.92	[kg]
Vertical component of seismic increase	-8.85	[kg]

<u>Combination n° 18 SLE (Frequent) - Vert. positive e.quake</u>

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	116.57	[kg]
Inclinazione linea di rottura in condizioni sismiche	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	116.01	[kg]
Vertical component of seismic increase	-11.42	[kg]

<u>Combination n° 19 SLE (Frequent) - Vert. Negative e.quake</u>

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	90.36	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	89.92	[kg]
Vertical component of seismic increase	-8.85	[kg]

<u>Combination n° 20 SLE (Rare) - Vert. Positive e.quake</u>

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]

Seismic increase of thrust	116.57	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	116.01	[kg]
Vertical component of seismic increase	-11.42	[kg]

Combination n° 21 SLE (Rare) - Vert. Negative e.quake

Static push	1640.17	[kg]
Inclination of break line in static conditions	0.00	[°]
Application point of thrust	X=0.00 [m] - Y=-2.66 [m]	
Horizontal component of static thrust	1632.28	[kg]
Vertical component of static thrust	-160.69	[kg]
Inclination of thrust compare to the horizontal line	-5.62	[°]
Seismic increase of thrust	90.36	[kg]
Inclination of break line in seismic condition	0.00	[°]
Application point of seismic thrust	X=0.00 [m] - Y=-1.40 [m]	
Horizontal Component of seismic increase	89.92	[kg]
Vertical component of seismic increase	-8.85	[kg]

5.6 Results

Combination n° 1 SLU (Case A1-M1)

Risults in X direction	2121.96	[kg]
Risults in Y direction	36415.15	[kg]
Normal stress on the laying of the foundation	36415.15	[kg]
Shear stress on the laying of the foundation	2121.96	[kg]
Eccentricity respect to the center of gravity of foundation	-0.911	[m]
Resultant in foundation	36476.93	[m]
Inclination of resultant (respect to the horizontal)	3.33	[°]
Limit load on foundation	190242.11	[kg]

Ground tensions

Foundation lenght reagent	2.82	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	2.584	[kg/cmq]

Factors for the calculation of the bearing capacity

$N_c = 46.12$ $N'_c = 42.77$
$N_q = 33.30$ $N'_q = 30.87$
$N_\gamma = 37.15$ $N'_\gamma = 30.41$

Combination n° 2 SLU (Case A2-M2)

Risults in X direction	2434.26	[kg]
Risults in Y direction	36415.15	[kg]
Normal stress on the laying of the foundation	36415.15	[kg]
Shear stress on the laying of the foundation	2434.26	[kg]
Eccentricity respect to the center of gravity of foundation	-0.853	[m]
Resultant in foundation	27869.27	[m]
Inclination of resultant (respect to the horizontal)	5.01	[°]
Limit load on foundation	63005.81	[kg]

Ground tensions

Foundation lenght reagent	2.99	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]

Ground pressure to the edge upstream	1.857	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c =28.42	N'_c = 25.35
N_q =16.92	N'_q = 15.09
N_γ = 13.82	N'_γ = 9.49

Combination n° 3 EQU

Risults in X direction	2677.68	[kg]
Risults in Y direction	24912.56	[kg]
Overturning moment on the laying of the foundation	5843.00	[kgm]
Stabilize moment on the laying of the foundation	72341.15	[kgm]
Normal stress on the laying of the foundation	24912.56	[kg]
Shear stress on the laying of the foundation	2677.68	[kg]
Eccentricity respect to the center of gravity of foundation	-0.819	[m]
Resultant in foundation	25056.05	[m]
Inclination of resultant (respect to the horizontal)	6.13	[°]

Combination n° 5 SLU (Case A1-M1) - Vert. negative e.quake

Risults in X direction	4097.79	[kg]
Risults in Y direction	26956.38	[kg]
Normal stress on the laying of the foundation	26956.38	[kg]
Shear stress on the laying of the foundation	4097.79	[kg]
Eccentricity respect to the center of gravity of foundation	-0.712	[m]
Resultant in foundation	27266.06	[m]
Inclination of resultant (respect to the horizontal)	8.64	[°]
Limit load on foundation	159634.46	[kg]

Ground tensions

Foundation lenght reagent	3.41	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.580	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12	N'_c = 37.69
N_q = 33.30	N'_q = 27.21
N_γ = 37.15	N'_γ = 21.07

Combination n° 6 SLU (Case A1-M1) - Vert. Positive e.quake

Risults in X direction	4211.70	[kg]
Risults in Y direction	28968.70	[kg]
Normal stress on the laying of the foundation	28968.70	[kg]
Shear stress on the laying of the foundation	4211.70	[kg]
Eccentricity respect to the center of gravity of foundation	-0.721	[m]
Resultant in foundation	29273.26	[m]
Inclination of resultant (respect to the horizontal)	8.27	[°]
Limit load on foundation	162868.30	[kg]

Ground tensions

Foundation lenght reagent	3.39	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.711	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12	N'_c = 38.03
N_q = 33.30	N'_q = 27.46
N_γ = 37.15	N'_γ = 21.67

Combination n° 7 SLU (Case A2-M2) - Vert. positive e.quake

Risults in X direction	5176.53	[kg]
Risults in Y direction	28653.59	[kg]
Normal stress on the laying of the foundation	28653.59	[kg]
Shear stress on the laying of the foundation	5176.53	[kg]
Eccentricity respect to the center of gravity of foundation	-0.642	[m]
Resultant in foundation	29117.43	[m]
Inclination of resultant (respect to the horizontal)	10.24	[°]
Limit load on foundation	46956.66	[kg]

Ground tensions

Lunghezza fondazione reagente	3.62	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.582	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 28.42	N'_c = 22.32
N_q = 16.92	N'_q = 13.29
N_γ = 13.82	N'_γ = 5.84

Combination n° 8 SLU (Case A2-M2) - Vert. negative e.quake

Risults in X direction	5005.76	[kg]
Risults in Y direction	26658.79	[kg]
Normal stress on the laying of the foundation	26658.79	[kg]
Shear stress on the laying of the foundation	5005.76	[kg]
Eccentricity respect to the center of gravity of foundation	-0.634	[m]
Resultant in foundation	27124.69	[m]
Inclination of resultant (respect to the horizontal)	10.63	[°]
Limit load on foundation	45323.63	[kg]

Ground tensions

Foundation lenght reagent	3.65	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.462	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 28.42	N'_c = 22.10
N_q = 16.92	N'_q = 13.16
N_γ = 13.82	N'_γ = 5.60

Combination n° 9 EQU - Vert. Negative e.quake

Risults in X direction	5005.76	[kg]
Risults in Y direction	26658.79	[kg]
Overturning moment on the laying of the foundation	14128.06	[kgm]
Stabilize moment on the laying of the foundation	80361.28	[kgm]
Normal stress on the laying of the foundation	26658.79	[kg]
Shear stress on the laying of the foundation	5005.76	[kg]
Eccentricity respect to the center of gravity of foundation	-0.634	[m]
Resultant in foundation	27124.69	[m]
Inclination of resultant (respect to the horizontal)	10.63	[°]

Combination n° 10 EQU - Vert. Positive e.quake

Risults in X direction	5176.53	[kg]
Risults in Y direction	28653.59	[kg]
Overturning moment on the laying of the foundation	11830.97	[kgm]
Stabilize moment on the laying of the foundation	83245.68	[kgm]
Normal stress on the laying of the foundation	28653.59	[kg]
Shear stress on the laying of the foundation	5176.53	[kg]
Eccentricity respect to the center of gravity of foundation	-0.642	[m]
Resultant in foundation	29117.43	[m]
Inclination of resultant (respect to the horizontal)	10.24	[°]

Combination n° 13 SLE (Almost Permanent)

Risults in X direction	1632.28	[kg]
Risults in Y direction	28011.66	[kg]
Normal stress on the laying of the foundation	28011.66	[kg]
Shear stress on the laying of the foundation	1632.28	[kg]
Eccentricity respect to the center of gravity of foundation	-0.912	[m]
Resultant in foundation	28059.17	[m]
Inclination of resultant (respect to the horizontal)	3.33	[°]
Limit load on foundation	189978.24	[kg]

Ground tensions

Foundation lenght reagent	2.81	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.991	[kg/cmq]

Factors for the calculation of the bearing capacity

$N_c = 46.12$ $N'_c = 42.77$
$N_q = 33.30$ $N'_q = 30.87$
$N_\gamma = 37.15$ $N'_\gamma = 30.41$

Combination n° 14 SLE (Frequent)

Risults in X direction	1632.28	[kg]
Risults in Y direction	28011.66	[kg]
Normal stress on the laying of the foundation	28011.66	[kg]

Shear stress on the laying of the foundation	1632.28	[kg]
Eccentricity respect to the center of gravity of foundation	-0.912	[m]
Resultant in foundation	28059.17	[m]
Inclination of resultant (respect to the horizontal)	3.33	[°]
Limit load on foundation	189978.24	[kg]

Ground tensions

Foundation lenght reagent	2.81	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.991	[kg/cmq]

Factors for the calculation of the bearing capacity

$N_c = 46.12$ $N'_c = 42.77$

$N_q = 33.30$ $N'_q = 30.87$

$N_\gamma = 37.15$ $N'_\gamma = 30.41$

Combination n° 15 SLE (Rare)

Risults in X direction	1632.28	[kg]
Risults in Y direction	28011.66	[kg]
Normal stress on the laying of the foundation	28011.66	[kg]
Shear stress on the laying of the foundation	1632.28	[kg]
Eccentricity respect to the center of gravity of foundation	-0.912	[m]
Resultant in foundation	28059.17	[m]
Inclination of resultant (respect to the horizontal)	3.33	[°]
Limit load on foundation	189978.24	[kg]

Ground tensions

Foundation lenght reagent	2.81	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.991	[kg/cmq]

Factors for the calculation of the bearing capacity

$N_c = 46.12$ $N'_c = 42.77$

$N_q = 33.30$ $N'_q = 30.87$

$N_\gamma = 37.15$ $N'_\gamma = 30.41$

Combination n° 16 SLE (Almost Permanent) - Vert. Positive e.quake

Risults in X direction	2198.51	[kg]
Risults in Y direction	28225.35	[kg]
Normal stress on the laying of the foundation	28225.35	[kg]
Shear stress on the laying of the foundation	2198.51	[kg]
Eccentricity respect to the center of gravity of foundation	-0.869	[m]
Resultant in foundation	28310.84	[m]
Inclination of resultant (respect to the horizontal)	4.45	[°]
Limit load on foundation	184851.29	[kg]

Ground tensions

Foundation lenght reagent	2.94	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.919	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12 N'_c = 41.67
N_q = 33.30 N'_q = 30.08
N_γ = 37.15 N'_γ = 28.30

Combination n° 17 SLE (Almost Permanent) - Vert. negative e.quake

Risults in X direction	2172.42	[kg]
Risults in Y direction	27777.69	[kg]
Normal stress on the laying of the foundation	27777.69	[kg]
Shear stress on the laying of the foundation	2172.42	[kg]
Eccentricity respect to the center of gravity of foundation	-0.870	[m]
Resultant in foundation	27862.52	[m]
Inclination of resultant (respect to the horizontal)	4.47	[°]
Limit load on foundation	184536.20	[kg]

Ground tensions

Foundation lenght reagent	2.94	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.889	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12 N'_c = 41.65
N_q = 33.30 N'_q = 30.07
N_γ = 37.15 N'_γ = 28.27

Combination n° 18 SLE (Frequent) - Vert. Positive e.quake

Risults in X direction	2198.51	[kg]
Risults in Y direction	28225.35	[kg]
Normal stress on the laying of the foundation	28225.35	[kg]
Shear stress on the laying of the foundation	2198.51	[kg]
Eccentricity respect to the center of gravity of foundation	-0.869	[m]
Resultant in foundation	28310.84	[m]
Inclination of resultant (respect to the horizontal)	4.45	[°]
Limit load on foundation	184851.29	[kg]

Ground tensions

Foundation lenght reagent	2.94	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.919	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12 N'_c = 41.67
N_q = 33.30 N'_q = 30.08
N_γ = 37.15 N'_γ = 28.30

Combination n° 19 SLE (Frequent) - Vert. Negative e.quake

Risults in X direction	2172.42	[kg]
Risults in Y direction	27777.69	[kg]
Normal stress on the laying of the foundation	27777.69	[kg]
Shear stress on the laying of the foundation	2172.42	[kg]
Eccentricity respect to the center of gravity of foundation	-0.870	[m]
Resultant in foundation	27862.52	[m]
Inclination of resultant (respect to the horizontal)	4.47	[°]
Limit load on foundation	184536.20	[kg]

Ground tensions

Foundation lenght reagent	2.94	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.889	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12 N'_c = 41.65
N_q = 33.30 N'_q = 30.07
N_γ = 37.15 N'_γ = 28.27

Combination n° 20 SLE (Rar) - Vert. Positive e.quake

Risults in X direction	2198.51	[kg]
Risults in Y direction	28225.35	[kg]
Normal stress on the laying of the foundation	28225.35	[kg]
Shear stress on the laying of the foundation	2198.51	[kg]
Eccentricity respect to the center of gravity of foundation	-0.869	[m]
Resultant in foundation	28310.84	[m]
Inclination of resultant (respect to the horizontal)	4.45	[°]
Limit load on foundation	184851.29	[kg]

Ground tensions

Foundation lenght reagent	2.94	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.919	[kg/cmq]

Factors for the calculation of the bearing capacity

N_c = 46.12 N'_c = 41.67
N_q = 33.30 N'_q = 30.08
N_γ = 37.15 N'_γ = 28.30

Combination n° 21 SLE (Rare) - Vert. Negative e.quake

Risults in X direction	2172.42	[kg]
Risults in Y direction	27777.69	[kg]
Normal stress on the laying of the foundation	27777.69	[kg]
Shear stress on the laying of the foundation	2172.42	[kg]
Eccentricity respect to the center of gravity of foundation	-0.870	[m]
Resultant in foundation	27862.52	[m]
Inclination of resultant (respect to the horizontal)	4.47	[°]
Limit load on foundation	184536.20	[kg]

Ground tensions

Foundation lenght reagent	2.94	[m]
Ground pressure to the edge downstream	0.000	[kg/cmq]
Ground pressure to the edge upstream	1.889	[kg/cmq]

Factors for the calculation of the bearing capacity

$N_c = 46.12$ $N'_c = 41.65$
$N_q = 33.30$ $N'_q = 30.07$
$N_\gamma = 37.15$ $N'_\gamma = 28.27$

4.7 Global stability of reinforced earth + ground

Symbols and sign conventions adopted

The x-axis X are considered positive upstream
The ordinates Y are considered positive upward
Origin at the head of reinforced earth (edge to ground)
Str ID Strip
W strip weight expressed in [kg]
α angle between the base of the strip and the horizontal expressed in [°] (negative clockwise)
ϕ friction angle of the soil along the base of the strip
c ground cohesion along the base of the strip expressed in [kg / sq.cm]
l base length of the strip expressed in [m]
u pore pressure along the base of the strip expressed in [kg / sq.cm]
N normal force at the base of the strip expressed in [kg]
T tangential stress at the base of the strip expressed in [kg]

Combination n° 4 STAB

Slidind surface n° 1 - Fs = 1.23

Str	**W**	**α**	**ϕ**	**c**	**l**	**u**	**N**	**T**
1	145	-33.57	29.26	0.00	0.59	0	120	55
2	408	-28.19	29.26	0.00	0.56	0	360	164
3	621	-23.20	29.26	0.00	0.54	0	570	261
4	788	-18.51	29.26	0.00	0.52	0	747	342
5	915	-14.05	29.26	0.00	0.51	0	887	406
6	1003	-9.77	29.26	0.00	0.50	0	988	452
7	1055	-4.26	29.26	0.00	0.49	0	1052	481
8	1180	0.27	29.26	0.00	0.49	0	1074	491
9	1959	4.43	29.26	0.00	0.49	0	1055	482

10	2827	8.64	29.26	0.00	0.50	0	995	455
11	3658	13.33	29.26	0.00	0.51	0	893	408
12	4374	18.84	29.26	0.00	0.52	0	751	343
13	4367	23.45	29.26	0.00	0.54	0	577	264
14	4157	28.33	29.26	0.00	0.56	0	372	170
15	3896	33.58	29.26	0.00	0.59	0	139	64
16	3553	39.25	26.56	0.00	0.64	0	189	77
17	3147	45.43	26.56	0.00	0.70	0	530	216
18	2620	52.16	26.56	0.00	0.80	0	703	287
19	1927	60.83	26.56	0.00	1.01	0	670	274
20	855	72.80	26.56	0.00	1.66	0	254	104

Combination n° 11 STAB - Vert. Positive e.quake

Sliding surface n° 1 - Fs = 1.10

Str	**W**	**α**	**ϕ**	**c**	**l**	**u**	**N**	**T**
1	145	-33.57	29.26	0.00	0.59	0	120	61
2	408	-28.19	29.26	0.00	0.56	0	360	183
3	621	-23.20	29.26	0.00	0.54	0	570	290
4	788	-18.51	29.26	0.00	0.52	0	747	381
5	915	-14.05	29.26	0.00	0.51	0	887	452
6	1003	-9.77	29.26	0.00	0.50	0	988	503
7	1055	-4.26	29.26	0.00	0.49	0	1052	536
8	1180	0.27	29.26	0.00	0.49	0	1074	547
9	1959	4.43	29.26	0.00	0.49	0	1055	537
10	2827	8.64	29.26	0.00	0.50	0	995	507
11	3658	13.33	29.26	0.00	0.51	0	893	455
12	4374	18.84	29.26	0.00	0.52	0	751	382
13	4367	23.45	29.26	0.00	0.54	0	577	294
14	4157	28.33	29.26	0.00	0.56	0	372	189
15	3896	33.58	29.26	0.00	0.59	0	139	71
16	3553	39.25	26.56	0.00	0.64	0	189	86
17	3147	45.43	26.56	0.00	0.70	0	530	241
18	2620	52.16	26.56	0.00	0.80	0	703	320
19	1927	60.83	26.56	0.00	1.01	0	670	305
20	855	72.80	26.56	0.00	1.66	0	254	116

Combination n° 12 STAB - Vert. Negative e.quake

Sliding surface n° 1 - Fs = 1.11

Str	**W**	**α**	**ϕ**	**c**	**l**	**u**	**N**	**T**
1	145	-33.57	29.26	0.00	0.59	0	120	61
2	408	-28.19	29.26	0.00	0.56	0	360	182
3	621	-23.20	29.26	0.00	0.54	0	570	288
4	788	-18.51	29.26	0.00	0.52	0	747	378
5	915	-14.05	29.26	0.00	0.51	0	887	449
6	1003	-9.77	29.26	0.00	0.50	0	988	500
7	1055	-4.26	29.26	0.00	0.49	0	1052	532
8	1180	0.27	29.26	0.00	0.49	0	1074	543
9	1959	4.43	29.26	0.00	0.49	0	1055	533
10	2827	8.64	29.26	0.00	0.50	0	995	503
11	3658	13.33	29.26	0.00	0.51	0	893	452
12	4374	18.84	29.26	0.00	0.52	0	751	380
13	4367	23.45	29.26	0.00	0.54	0	577	292
14	4157	28.33	29.26	0.00	0.56	0	372	188
15	3896	33.58	29.26	0.00	0.59	0	139	70
16	3553	39.25	26.56	0.00	0.64	0	189	85
17	3147	45.43	26.56	0.00	0.70	0	530	239
18	2620	52.16	26.56	0.00	0.80	0	703	317
19	1927	60.83	26.56	0.00	1.01	0	670	302
20	855	72.80	26.56	0.00	1.66	0	254	115

5.8 Internal checks

Reinforcements results

symbols used

N. ID reinforcement

z reinforcement height, expressed in [m]

Rinf reinforcement type used

Sf reinforcement stress expressed in [kg / m]

Ll free length expressed in [m]

Lf reinforcement foundation lenght expressed in [m]

Lt reinforcement total length expressed in [m]

Lrisv reinforcement flap length expressed in [m]

Combination n° 1 SLU (Case A1-M1)

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	reinforcement 1	33.38	1.48	2.22	3.70	1.50
2	-0.80	reinforcement 1	105.91	1.32	2.38	3.70	1.20
3	-1.20	reinforcement 1	137.38	1.15	2.55	3.70	1.20
4	-1.60	reinforcement 1	237.55	0.99	2.71	3.70	1.20
5	-2.00	reinforcement 1	242.69	0.82	2.88	3.70	1.20
6	-2.40	reinforcement 1	369.42	0.66	3.04	3.70	1.20
7	-2.80	reinforcement 1	349.35	0.49	3.21	3.70	1.20
8	-3.20	reinforcement 1	502.54	0.33	3.37	3.70	1.20
9	-3.60	reinforcement 1	454.69	0.16	3.54	3.70	1.20
10	-4.00	reinforcement 1	501.50	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	reinforcement 1	36.88	73.77	34.56	3.12
2	-0.80	reinforcement 1	25.01	50.02	10.89	6.42
3	-1.20	reinforcement 1	30.92	61.85	8.40	8.36
4	-1.60	reinforcement 1	25.39	50.77	4.86	6.35
5	-2.00	reinforcement 1	32.55	65.09	4.75	7.23
6	-2.40	reinforcement 1	26.00	52.00	3.12	5.12
7	-2.80	reinforcement 1	31.82	63.63	3.30	5.53
8	-3.20	reinforcement 1	24.73	49.47	2.30	3.85
9	-3.60	reinforcement 1	29.79	59.57	2.54	4.25
10	-4.00	reinforcement 1	28.94	57.89	2.30	3.86

Combination n° 2 SLU (Case A2-M2)

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	38.46	1.82	1.89	3.70	1.50
2	-0.80	1	121.75	1.61	2.09	3.70	1.20
3	-1.20	1	157.83	1.41	2.29	3.70	1.20
4	-1.60	1	272.85	1.21	2.49	3.70	1.20
5	-2.00	1	279.00	1.01	2.69	3.70	1.20
6	-2.40	1	425.88	0.81	2.89	3.70	1.20
7	-2.80	1	401.18	0.61	3.10	3.70	1.20
8	-3.20	1	577.03	0.40	3.30	3.70	1.20
9	-3.60	1	522.07	0.20	3.50	3.70	1.20
10	-4.00	1	575.81	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	16.76	33.51	30.00	1.67
2	-0.80	1	11.73	23.47	9.48	3.44
3	-1.20	1	14.89	29.78	7.31	4.48
4	-1.60	1	12.49	24.99	4.23	3.40
5	-2.00	1	16.47	32.94	4.14	3.87
6	-2.40	1	13.50	26.99	2.71	2.73
7	-2.80	1	16.83	33.66	2.88	2.96
8	-3.20	1	13.20	26.39	2.00	2.06
9	-3.60	1	15.96	31.91	2.21	2.28
10	-4.00	1	15.51	31.03	2.00	2.07

Combination n° 5 SLU (Case A1-M1) - Vert. negative e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	31.84	1.80	1.90	3.70	1.50
2	-0.80	1	97.56	1.60	2.10	3.70	1.20
3	-1.20	1	131.70	1.40	2.30	3.70	1.20
4	-1.60	1	218.67	1.20	2.50	3.70	1.20
5	-2.00	1	232.56	1.00	2.70	3.70	1.20
6	-2.40	1	339.97	0.80	2.90	3.70	1.20
7	-2.80	1	334.45	0.60	3.10	3.70	1.20
8	-3.20	1	462.22	0.40	3.30	3.70	1.20
9	-3.60	1	435.34	0.20	3.50	3.70	1.20
10	-4.00	1	481.28	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	25.51	51.02	36.23	2.52
2	-0.80	1	18.43	36.85	11.83	5.36
3	-1.20	1	22.42	44.85	8.76	6.71
4	-1.60	1	19.57	39.13	5.28	5.30
5	-2.00	1	24.77	49.54	4.96	5.80
6	-2.40	1	21.16	42.33	3.39	4.28
7	-2.80	1	25.26	50.51	3.45	4.44
8	-3.20	1	20.60	41.20	2.50	3.22
9	-3.60	1	23.92	47.84	2.65	3.42
10	-4.00	1	23.20	46.40	2.40	3.09

Combination n° 6 SLU (Case A1-M1) - Vert. Positive e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	33.26	1.78	1.93	3.70	1.50
2	-0.80	1	101.82	1.58	2.12	3.70	1.20
3	-1.20	1	138.79	1.38	2.32	3.70	1.20
4	-1.60	1	228.61	1.18	2.52	3.70	1.20
5	-2.00	1	245.33	0.99	2.72	3.70	1.20
6	-2.40	1	355.58	0.79	2.91	3.70	1.20
7	-2.80	1	352.90	0.59	3.11	3.70	1.20

8	-3.20	1	483.51	0.39	3.31	3.70	1.20
9	-3.60	1	459.47	0.20	3.50	3.70	1.20
10	-4.00	1	508.24	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	24.72	49.45	34.69	2.41
2	-0.80	1	17.83	35.66	11.33	5.13
3	-1.20	1	21.45	42.89	8.31	6.37
4	-1.60	1	18.83	37.67	5.05	5.07
5	-2.00	1	23.59	47.17	4.70	5.50
6	-2.40	1	20.28	40.55	3.25	4.09
7	-2.80	1	23.96	47.92	3.27	4.21
8	-3.20	1	19.70	39.40	2.39	3.08
9	-3.60	1	22.66	45.33	2.51	3.24
10	-4.00	1	21.97	43.94	2.27	2.93

Combination n° 7 SLU (Case A2-M2) - Vert. positive e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	47.85	2.16	1.54	3.70	1.50
2	-0.80	1	147.51	1.92	1.78	3.70	1.20
3	-1.20	1	199.97	1.68	2.02	3.70	1.20
4	-1.60	1	331.36	1.44	2.26	3.70	1.20
5	-2.00	1	353.89	1.20	2.50	3.70	1.20
6	-2.40	1	517.14	0.96	2.74	3.70	1.20
7	-2.80	1	508.81	0.72	2.98	3.70	1.20
8	-3.20	1	701.04	0.48	3.22	3.70	1.20
9	-3.60	1	662.45	0.24	3.46	3.70	1.20
10	-4.00	1	732.56	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	10.98	21.97	24.11	1.34
2	-0.80	1	8.25	16.51	7.82	2.84
3	-1.20	1	10.37	20.73	5.77	3.54
4	-1.60	1	9.33	18.66	3.48	2.80
5	-2.00	1	12.08	24.16	3.26	3.05
6	-2.40	1	10.70	21.41	2.23	2.25
7	-2.80	1	13.05	26.11	2.27	2.34
8	-3.20	1	10.80	21.60	1.65	1.70
9	-3.60	1	12.56	25.13	1.74	1.80
10	-4.00	1	12.19	24.39	1.58	1.62

Combination n° 8 SLU (Case A2-M2) - Vert. negative e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	45.72	2.20	1.51	3.70	1.50
2	-0.80	1	141.12	1.95	1.75	3.70	1.20
3	-1.20	1	189.33	1.71	1.99	3.70	1.20
4	-1.60	1	316.46	1.47	2.24	3.70	1.20

5	-2.00	1	334.73	1.22	2.48	3.70	1.20
6	-2.40	1	493.73	0.98	2.72	3.70	1.20
7	-2.80	1	481.14	0.73	2.97	3.70	1.20
8	-3.20	1	669.11	0.49	3.21	3.70	1.20
9	-3.60	1	626.27	0.24	3.46	3.70	1.20
10	-4.00	1	692.12	0.00	3.70	3.70	1.20

n°	**z**	**Rinf**	**Fs scor**	**Fs sfil**	**Fs traz**	**Fs risv**
1	-0.40	1	11.23	22.47	25.24	1.40
2	-0.80	1	8.48	16.95	8.18	2.96
3	-1.20	1	10.80	21.60	6.09	3.73
4	-1.60	1	9.67	19.34	3.65	2.93
5	-2.00	1	12.67	25.34	3.45	3.23
6	-2.40	1	11.16	22.32	2.34	2.36
7	-2.80	1	13.78	27.56	2.40	2.47
8	-3.20	1	11.31	22.61	1.72	1.78
9	-3.60	1	13.29	26.58	1.84	1.90
10	-4.00	1	12.91	25.81	1.67	1.72

Combination n° 13 SLE (Almost Permanent)

n°	**z**	**Rinf**	**Sf**	**Ll**	**Lf**	**Lt**	**Lrisv**
1	-0.40	1	25.68	1.48	2.22	3.70	1.50
2	-0.80	1	81.47	1.32	2.38	3.70	1.20
3	-1.20	1	105.68	1.15	2.55	3.70	1.20
4	-1.60	1	182.73	0.99	2.71	3.70	1.20
5	-2.00	1	186.69	0.82	2.88	3.70	1.20
6	-2.40	1	284.17	0.66	3.04	3.70	1.20
7	-2.80	1	268.73	0.49	3.21	3.70	1.20
8	-3.20	1	386.57	0.33	3.37	3.70	1.20
9	-3.60	1	349.76	0.16	3.54	3.70	1.20
10	-4.00	1	385.77	0.00	3.70	3.70	1.20

n°	**z**	**Rinf**	**Fs scor**	**Fs sfil**	**Fs traz**	**Fs risv**
1	-0.40	1	36.88	73.77	44.93	3.12
2	-0.80	1	25.01	50.02	14.16	6.42
3	-1.20	1	30.92	61.85	10.92	8.36
4	-1.60	1	25.39	50.77	6.31	6.35
5	-2.00	1	32.55	65.09	6.18	7.23
6	-2.40	1	26.00	52.00	4.06	5.12
7	-2.80	1	31.82	63.63	4.29	5.53
8	-3.20	1	24.73	49.47	2.98	3.85
9	-3.60	1	29.79	59.57	3.30	4.25
10	-4.00	1	28.94	57.89	2.99	3.86

Combination n° 14 SLE (Frequent)

n°	**z**	**Rinf**	**Sf**	**Ll**	**Lf**	**Lt**	**Lrisv**
1	-0.40	1	25.68	1.48	2.22	3.70	1.50

2	-0.80	1	81.47	1.32	2.38	3.70	1.20
3	-1.20	1	105.68	1.15	2.55	3.70	1.20
4	-1.60	1	182.73	0.99	2.71	3.70	1.20
5	-2.00	1	186.69	0.82	2.88	3.70	1.20
6	-2.40	1	284.17	0.66	3.04	3.70	1.20
7	-2.80	1	268.73	0.49	3.21	3.70	1.20
8	-3.20	1	386.57	0.33	3.37	3.70	1.20
9	-3.60	1	349.76	0.16	3.54	3.70	1.20
10	-4.00	1	385.77	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	36.88	73.77	44.93	3.12
2	-0.80	1	25.01	50.02	14.16	6.42
3	-1.20	1	30.92	61.85	10.92	8.36
4	-1.60	1	25.39	50.77	6.31	6.35
5	-2.00	1	32.55	65.09	6.18	7.23
6	-2.40	1	26.00	52.00	4.06	5.12
7	-2.80	1	31.82	63.63	4.29	5.53
8	-3.20	1	24.73	49.47	2.98	3.85
9	-3.60	1	29.79	59.57	3.30	4.25
10	-4.00	1	28.94	57.89	2.99	3.86

Combination n° 15 SLE (Rare)

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	25.68	1.48	2.22	3.70	1.50
2	-0.80	1	81.47	1.32	2.38	3.70	1.20
3	-1.20	1	105.68	1.15	2.55	3.70	1.20
4	-1.60	1	182.73	0.99	2.71	3.70	1.20
5	-2.00	1	186.69	0.82	2.88	3.70	1.20
6	-2.40	1	284.17	0.66	3.04	3.70	1.20
7	-2.80	1	268.73	0.49	3.21	3.70	1.20
8	-3.20	1	386.57	0.33	3.37	3.70	1.20
9	-3.60	1	349.76	0.16	3.54	3.70	1.20
10	-4.00	1	385.77	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	36.88	73.77	44.93	3.12
2	-0.80	1	25.01	50.02	14.16	6.42
3	-1.20	1	30.92	61.85	10.92	8.36
4	-1.60	1	25.39	50.77	6.31	6.35
5	-2.00	1	32.55	65.09	6.18	7.23
6	-2.40	1	26.00	52.00	4.06	5.12
7	-2.80	1	31.82	63.63	4.29	5.53
8	-3.20	1	24.73	49.47	2.98	3.85
9	-3.60	1	29.79	59.57	3.30	4.25
10	-4.00	1	28.94	57.89	2.99	3.86

Combination n° 16 SLE (Almost Permanent) - Vert. Positive e .1uake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	28.26	1.55	2.15	3.70	1.50
2	-0.80	1	86.81	1.38	2.33	3.70	1.20
3	-1.20	1	113.77	1.21	2.50	3.70	1.20
4	-1.60	1	193.58	1.03	2.67	3.70	1.20
5	-2.00	1	200.29	0.86	2.84	3.70	1.20
6	-2.40	1	300.53	0.69	3.01	3.70	1.20
7	-2.80	1	287.84	0.52	3.18	3.70	1.20
8	-3.20	1	408.44	0.34	3.36	3.70	1.20
9	-3.60	1	374.39	0.17	3.53	3.70	1.20
10	-4.00	1	413.16	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	32.53	65.06	40.83	2.84
2	-0.80	1	22.90	45.80	13.29	6.02
3	-1.20	1	28.15	56.30	10.14	7.77
4	-1.60	1	23.58	47.16	5.96	5.99
5	-2.00	1	30.03	60.07	5.76	6.74
6	-2.40	1	24.46	48.93	3.84	4.84
7	-2.80	1	29.64	59.27	4.01	5.16
8	-3.20	1	23.39	46.78	2.83	3.64
9	-3.60	1	27.82	55.65	3.08	3.97
10	-4.00	1	27.03	54.05	2.79	3.60

Combination n° 17 SLE (Almost Permanent) - Vert. negative e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	27.94	1.55	2.15	3.70	1.50
2	-0.80	1	85.84	1.38	2.33	3.70	1.20
3	-1.20	1	112.16	1.21	2.50	3.70	1.20
4	-1.60	1	191.32	1.03	2.67	3.70	1.20
5	-2.00	1	197.38	0.86	2.84	3.70	1.20
6	-2.40	1	296.98	0.69	3.01	3.70	1.20
7	-2.80	1	283.65	0.52	3.18	3.70	1.20
8	-3.20	1	403.60	0.34	3.36	3.70	1.20
9	-3.60	1	368.91	0.17	3.53	3.70	1.20
10	-4.00	1	407.03	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	32.91	65.81	41.30	2.87
2	-0.80	1	23.16	46.32	13.44	6.09
3	-1.20	1	28.56	57.11	10.29	7.88
4	-1.60	1	23.86	47.71	6.03	6.06
5	-2.00	1	30.48	60.95	5.85	6.84
6	-2.40	1	24.76	49.51	3.89	4.90
7	-2.80	1	30.08	60.15	4.07	5.24
8	-3.20	1	23.67	47.34	2.86	3.69
9	-3.60	1	28.24	56.48	3.13	4.03
10	-4.00	1	27.43	54.87	2.83	3.65

Combination n° 18 SLE (Frequent) - Vert. positive e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	28.26	1.55	2.15	3.70	1.50
2	-0.80	1	86.81	1.38	2.33	3.70	1.20
3	-1.20	1	113.77	1.21	2.50	3.70	1.20
4	-1.60	1	193.58	1.03	2.67	3.70	1.20
5	-2.00	1	200.29	0.86	2.84	3.70	1.20
6	-2.40	1	300.53	0.69	3.01	3.70	1.20
7	-2.80	1	287.84	0.52	3.18	3.70	1.20
8	-3.20	1	408.44	0.34	3.36	3.70	1.20
9	-3.60	1	374.39	0.17	3.53	3.70	1.20
10	-4.00	1	413.16	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	32.53	65.06	40.83	2.84
2	-0.80	1	22.90	45.80	13.29	6.02
3	-1.20	1	28.15	56.30	10.14	7.77
4	-1.60	1	23.58	47.16	5.96	5.99
5	-2.00	1	30.03	60.07	5.76	6.74
6	-2.40	1	24.46	48.93	3.84	4.84
7	-2.80	1	29.64	59.27	4.01	5.16
8	-3.20	1	23.39	46.78	2.83	3.64
9	-3.60	1	27.82	55.65	3.08	3.97
10	-4.00	1	27.03	54.05	2.79	3.60

Combination n° 19 SLE (Frequent) - Vert. Negative e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	27.94	1.55	2.15	3.70	1.50
2	-0.80	1	85.84	1.38	2.33	3.70	1.20
3	-1.20	1	112.16	1.21	2.50	3.70	1.20
4	-1.60	1	191.32	1.03	2.67	3.70	1.20
5	-2.00	1	197.38	0.86	2.84	3.70	1.20
6	-2.40	1	296.98	0.69	3.01	3.70	1.20
7	-2.80	1	283.65	0.52	3.18	3.70	1.20
8	-3.20	1	403.60	0.34	3.36	3.70	1.20
9	-3.60	1	368.91	0.17	3.53	3.70	1.20
10	-4.00	1	407.03	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	32.91	65.81	41.30	2.87
2	-0.80	1	23.16	46.32	13.44	6.09
3	-1.20	1	28.56	57.11	10.29	7.88
4	-1.60	1	23.86	47.71	6.03	6.06
5	-2.00	1	30.48	60.95	5.85	6.84
6	-2.40	1	24.76	49.51	3.89	4.90
7	-2.80	1	30.08	60.15	4.07	5.24

8	-3.20	1	23.67	47.34	2.86	3.69
9	-3.60	1	28.24	56.48	3.13	4.03
10	-4.00	1	27.43	54.87	2.83	3.65

Combination n° 20 SLE (Rar) - Vert. positive e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	28.26	1.55	2.15	3.70	1.50
2	-0.80	1	86.81	1.38	2.33	3.70	1.20
3	-1.20	1	113.77	1.21	2.50	3.70	1.20
4	-1.60	1	193.58	1.03	2.67	3.70	1.20
5	-2.00	1	200.29	0.86	2.84	3.70	1.20
6	-2.40	1	300.53	0.69	3.01	3.70	1.20
7	-2.80	1	287.84	0.52	3.18	3.70	1.20
8	-3.20	1	408.44	0.34	3.36	3.70	1.20
9	-3.60	1	374.39	0.17	3.53	3.70	1.20
10	-4.00	1	413.16	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	32.53	65.06	40.83	2.84
2	-0.80	1	22.90	45.80	13.29	6.02
3	-1.20	1	28.15	56.30	10.14	7.77
4	-1.60	1	23.58	47.16	5.96	5.99
5	-2.00	1	30.03	60.07	5.76	6.74
6	-2.40	1	24.46	48.93	3.84	4.84
7	-2.80	1	29.64	59.27	4.01	5.16
8	-3.20	1	23.39	46.78	2.83	3.64
9	-3.60	1	27.82	55.65	3.08	3.97
10	-4.00	1	27.03	54.05	2.79	3.60

Combination n° 21 SLE (Rare) - Vert. Negative e.quake

n°	z	Rinf	Sf	Ll	Lf	Lt	Lrisv
1	-0.40	1	27.94	1.55	2.15	3.70	1.50
2	-0.80	1	85.84	1.38	2.33	3.70	1.20
3	-1.20	1	112.16	1.21	2.50	3.70	1.20
4	-1.60	1	191.32	1.03	2.67	3.70	1.20
5	-2.00	1	197.38	0.86	2.84	3.70	1.20
6	-2.40	1	296.98	0.69	3.01	3.70	1.20
7	-2.80	1	283.65	0.52	3.18	3.70	1.20
8	-3.20	1	403.60	0.34	3.36	3.70	1.20
9	-3.60	1	368.91	0.17	3.53	3.70	1.20
10	-4.00	1	407.03	0.00	3.70	3.70	1.20

n°	z	Rinf	Fs scor	Fs sfil	Fs traz	Fs risv
1	-0.40	1	32.91	65.81	41.30	2.87
2	-0.80	1	23.16	46.32	13.44	6.09
3	-1.20	1	28.56	57.11	10.29	7.88
4	-1.60	1	23.86	47.71	6.03	6.06

5	-2.00	1	30.48	60.95	5.85	6.84
6	-2.40	1	24.76	49.51	3.89	4.90
7	-2.80	1	30.08	60.15	4.07	5.24
8	-3.20	1	23.67	47.34	2.86	3.69
9	-3.60	1	28.24	56.48	3.13	4.03
10	-4.00	1	27.43	54.87	2.83	3.65

5.9 Composed checks

Global stability of reinforced earth

Symbols and sign conventions adopted

The x-axis X are considered positive upstream
The ordinates Y are considered positive upward
Origin at the head of reinforced earth (edge to ground)
Str ID Strip
W strip weight expressed in [kg]
α angle between the base of the strip and the horizontal expressed in [°] (positive clockwise)
ϕ soil friction angle along the base of the strip
c ground cohesion along the base of the strip expressed in [kg / sq.cm]
b is the width of the strip expressed in [m]
u pore pressure along the base of the strip expressed in [kg / sq.cm]
N normal force at the base of the strip expressed in [kg]
T tangential stress at the base of the strip expressed in [kg]

Combination n° 4 STAB

Sliding surface n° 11 - Fs = 1.40

Str	**W**	**α**	**ϕ**	**c**	**b**	**u**	**N**	**T**
1	47	12.38	26.56	0.00	0.17	0	46	16
2	141	13.77	26.56	0.00	0.17	0	137	49
3	232	16.36	26.56	0.00	0.17	0	222	79
4	321	19.24	26.56	0.00	0.18	0	303	108
5	407	20.87	26.56	0.00	0.18	0	380	135
6	490	24.32	26.56	0.00	0.18	0	447	159
7	571	26.68	26.56	0.00	0.19	0	510	182
8	649	28.68	26.56	0.00	0.19	0	569	203
9	723	32.28	26.56	0.00	0.20	0	611	218
10	794	34.95	26.56	0.00	0.20	0	651	232
11	861	37.47	26.56	0.00	0.21	0	683	243
12	923	40.24	26.56	0.00	0.22	0	705	251
13	928	44.22	26.56	0.00	0.23	0	665	237
14	874	47.40	26.56	0.00	0.25	0	592	211
15	813	50.89	26.56	0.00	0.26	0	513	183
16	744	54.73	26.56	0.00	0.29	0	430	153
17	663	58.94	26.56	0.00	0.32	0	342	122
18	566	63.94	26.56	0.00	0.38	0	248	89
19	441	70.12	26.56	0.00	0.49	0	151	54
20	241	81.74	26.56	0.00	1.16	0	35	12

Combination n° 11 STAB - Vert. Positive e.quake

Sliding surface n° 16 - Fs = 1.36

Str	**W**	**α**	**ϕ**	**c**	**b**	**u**	**N**	**T**
1	69	3.82	26.56	0.00	0.19	0	69	26
2	208	5.67	26.56	0.00	0.20	0	207	76
3	342	8.24	26.56	0.00	0.20	0	339	125
4	473	11.99	26.56	0.00	0.20	0	463	170
5	601	14.00	26.56	0.00	0.20	0	583	215
6	724	17.08	26.56	0.00	0.20	0	692	255
7	843	20.55	26.56	0.00	0.21	0	790	291
8	959	22.88	26.56	0.00	0.21	0	883	325
9	1069	25.92	26.56	0.00	0.22	0	961	354
10	1175	29.82	26.56	0.00	0.22	0	1019	375
11	1239	32.64	26.56	0.00	0.23	0	1043	384
12	1196	35.70	26.56	0.00	0.24	0	971	358
13	1140	39.18	26.56	0.00	0.25	0	884	325
14	1077	43.49	26.56	0.00	0.27	0	781	288
15	1004	47.29	26.56	0.00	0.29	0	681	251
16	921	51.46	26.56	0.00	0.31	0	575	212
17	823	56.03	26.56	0.00	0.35	0	462	170
18	704	61.83	26.56	0.00	0.41	0	334	123
19	549	68.16	26.56	0.00	0.52	0	206	76
20	300	81.02	26.56	0.00	1.25	0	47	17

Combination n° 12 STAB - Vert. Negative e.quake

Sliding surface n° 11 - Fs = 1.31

Str	**W**	**α**	**ϕ**	**c**	**b**	**u**	**N**	**T**
1	47	12.38	26.56	0.00	0.17	0	46	18
2	141	13.77	26.56	0.00	0.17	0	137	52
3	232	16.36	26.56	0.00	0.17	0	222	85
4	321	19.24	26.56	0.00	0.18	0	303	115
5	407	20.87	26.56	0.00	0.18	0	380	145
6	490	24.32	26.56	0.00	0.18	0	447	170
7	571	26.68	26.56	0.00	0.19	0	510	195
8	649	28.68	26.56	0.00	0.19	0	569	217
9	723	32.28	26.56	0.00	0.20	0	611	233
10	794	34.95	26.56	0.00	0.20	0	651	248
11	861	37.47	26.56	0.00	0.21	0	683	261
12	923	40.24	26.56	0.00	0.22	0	705	269
13	928	44.22	26.56	0.00	0.23	0	665	254
14	874	47.40	26.56	0.00	0.25	0	592	226
15	813	50.89	26.56	0.00	0.26	0	513	196
16	744	54.73	26.56	0.00	0.29	0	430	164
17	663	58.94	26.56	0.00	0.32	0	342	130
18	566	63.94	26.56	0.00	0.38	0	248	95
19	441	70.12	26.56	0.00	0.49	0	151	58
20	241	81.74	26.56	0.00	1.16	0	35	13

6. CONCLUSION

Type of analysis conducted

The analysis and verifications are conducted with the aid of an automatic calculation code .
The calculation of the lands armed is performed according to the following phases:
- Calculation of the earth thrust;
- Overturning check ;
- Sliding check the subfloor ;
- Verification of stability of complex foundation soil (limit load);
- Verification of global stability ;
- Check the work against potential failure surfaces inside the reinforced earth . In particular, an analysis is performed of internal stability or local (tieback) which allows to obtain a homogeneous distribution of the tensions in the reinforcements , and a global analysis (compound) that ensures the overall stability and , in particular , the existence of reinforcements of sufficient length to ensure its anchor in a portion of the soil stable .

The analysis under seismic actions is conducted by the method of static equivalent in accordance with the provisions of Chapter 7 of the DM 14/01/2008 .
The load combinations used are exhaustive in relation to the heaviest load scenarios where the work will be subject .

Reliability of computer codes

A careful preliminary examination of the documentation supplied with the software made it possible to assess their reliability. The documentation provided by the manufacturer of the software will contain a full description of the theoretical basis of the algorithms used and the identification of areas of application. The company that Aztec Computers srl has checked the reliability and robustness of the calculation code through a significant number of test cases in which the results of numerical analysis were compared with the theoretical solutions.

Presentation of the results

The structural calculations presents the calculation data as to ensure legibility, the correct interpretation and reproducibility. The relationship of calculation illustrates comprehensively the input data and the analysis results in tabular form.

General information about processing

The software provides a series of automatic controls that allow the detection of modeling errors, respect to geometric limitations and armature and the presence of elements not verified. The computer code allows you to view and control, both in graphical and tabular form, the data of the structural model, in order to have a clear vision of the correct behavior of the structural model.

Reasoned judgment on the acceptability of results

The processing results were subjected to controls by the subscribed user of the software. This evaluation included the comparison with the results of simple calculations, performed with traditional methods. Furthermore, on the basis of considerations relating to the stress and deformation were determined, we evaluated the validity of the choices made in the schematic and modeling of the structure and actions.
Based on the above, I hereby assert that the processing is correct and suitable to the specific case, therefore, the results of calculation are to be considered valid and acceptable.

The parametric study results showed that on the design approach 1, the combination C2 is always cautionary than C1.
With that conclusion, it is confirmed that the project of a flexibly work, where it is believed that the major uncertainties reside in the choice of geotechnical parameters, it is more appropriate to move towards the approach DA1.C2 in which the partial factors are applied to the parameters rather than the actions.
This is among other things also the meaning of the Ministerial Circular updated on 7 March 2008:

"Bozza di istruzioni per l'applicazione delle norme tecniche per le costruzioni di cui al D.M. 14 gennaio 2008" che suggeriscono appunto:
"Nelle verifiche agli stati limite ultimi per il dimensionamento geotecnico dei muri e delle paratie (GEO), si considera lo sviluppo di meccanismi di collasso determinati dalla mobilitazione della resistenza del terreno. L'analisi può essere condotta con la Combinazione 2 (A2+M2+R1), nella quale i parametri di resistenza del terreno sono ridotti tramite i coefficienti parziali del gruppo M2, i coefficienti γ_R sulla resistenza globale (R1) sono unitari e le sole azioni variabili sono amplificate con i coefficienti del gruppo A2. I parametri di resistenza di progetto sono perciò inferiori a quelli caratteristici e di conseguenza il valore di progetto della spinta attiva è maggiore, e quello della resistenza passiva è minore, dei corrispondenti valori caratteristici.
Nelle verifiche STR si considerano gli stati limite ultimi per raggiungimento della resistenza negli elementi strutturali. L'analisi può essere svolta utilizzando la Combinazione 1 (A1+M1+R1), nella quale i coefficienti sui parametri di resistenza *del terreno (M1) e sulla resistenza globale del sistema (R1) sono unitari, mentre le azioni permanenti e variabili sono amplificate mediante i coefficienti parziali del gruppo A1. In questo caso, i coefficienti parziali amplificativi delle azioni possono applicarsi direttamente alle sollecitazioni, calcolate con i valori caratteristici delle azioni e delle resistenze."*

Regarding the seismic conditions, the national legislation undoubtedly introduces innovative elements, and more appropriate to the level of scientific knowledge, in the characterization of seismic hazard of the sites and in the evaluation of the role of local geotechnical conditions. With regard to the effects of amplification, the classification of subsoils must be verified over the years in real works falling under the complex reality of the Italian land.

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