GAWAC 3.0

Gabion Wall Design

Reference Manual

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MACCAFERRI

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Summary

Introduction

The Gawac GSC 2019 Program has been developed to provide engineers with a rapid and efficient tool to conduct the stability analysis of gabion retaining walls. The program allows the check under many different situations (geometry, surcharge loads, etc.) which may occur during the design process. The program uses the Limit Equilibrium and the theories of Rankine, Coulomb, Meyerhof, Hansen and Bishop (optimized through the Simplex Minimizer Algorithm) to check the global stability of the soil/structure. The program requires the user to provide the problem data and perform the analysis commands. Following this process, the program will print a report containing the problem data and the analysis results.

The mechanical characteristics of gabions manufactured by the MACCAFERRI Group are taken into consideration by the program; therefore, the results of the calculation will not be realistic in the case of using other types of materials.

To facilitate and improve the comprehension of the program, it has been provided with a graphic interface integrated with a pull-down menu and tool bar, which allows the user to check the results of the data input in a simple and direct way. With the graphic interface it is always possible to check the cross section of the wall, the uphill soil geometry, the foundation, and the external surcharge loads. Using either the View menu commands or the tool bar, the view can be enlarged, decreased, and moved. In addition, the user may edit the geometry using the mouse like in a CAD program.

The programs main calculation hypothesis considers the problem to have a planar configuration, requiring only the cross-section dimensions for the analysis. In such a hypothesis the effects caused by variations in the loads or in the soil geometry in the direction perpendicular to the plane are neglected. On the other hand, an analysis of the problem which considers the above effects would be more complete, but it would also make the calculation, and the data required to describe the problem, more complex. Experience proves that except cases, the analysis of plane surfaces with respect to three-dimensional ones provides more conservative results thus a higher factor of safety.

The theories mentioned here have some limitations regarding their use in special situations. In such cases, since the program can deal with a huge variety of situations, the user may choose to perform complementary comparisons and analysis to overcome these limitations. For such situations, it is recommended the user contact Technical Department for assistance.

1. Retaining walls

Retaining structures are civil works built with the purpose of providing stability against the rupture of land or rock masses. They are structures that provide support for the mass of soil and avoid slipping caused by their own weight or by external loads. Typical examples of retaining structures are the retaining walls, the pilling and the diaphragm walls. The constructive process and the materials used in the mentioned structures are very different from each other, they are all constructed to contain the possible rupture of the massif, by supporting lateral pressures exerted by it.

Retaining structures are among the oldest human constructions, accompanying civilization from the earliest stone constructions of prehistory. However, the design using theoretical models, only developed from the eighteenth century. In 1773, Coulomb presented his work "Essai sur une des règles de maximis et minimis à quelques problèmes de statique, relatifs à l'achitecture". In one of the chapters of this work Coulomb deals with the determination of the lateral thrust applied by the soil on a support structure. This determination is the most important step in the design of a support structure. Coulomb's work still constitutes one of the main bases of the current methods of dimensioning the retaining walls. Even with the development of modern Soil Mechanics, the Coulomb model continues to be widely applied. The original Coulomb article is reproduced in Heyman's book, along with a historical analysis of the development of earthbound determination theories.

The analysis of a retaining structure consists of the analysis of the balance of the set formed by the soil mass and the structure itself. This equilibrium is affected by the characteristics of resistance, deformability, permeability and by the proper weight of these two elements, besides the conditions that govern the interaction between them. These conditions make the system very complex and there is therefore a need to adopt simplified theoretical models that make the analysis possible. These models should consider the characteristics of materials that influence overall behavior, as well as geometry and local conditions.

On the solid side must be considered their own weight, strength, deformability and geometry. In addition, data on local drainage conditions and external loads applied to the soil are required. On the side of the structure must be considered its geometry, material employed, and the constructive system adopted. Finally, from the

In view of the interaction, the characteristics of the interfaces between the soil and the structure, in addition to the constructive sequence, should be considered in the analysis.

2. Soil shear resistance

The shear strength can be defined as the maximum value that the shear stress can reach along any plane within the mass without any rupture of the soil structure. Since a large part of this resistance comes from the friction between the soil particles, it depends on the normal stress acting on this plane. On the other hand, most thrust problems can be approximated to a flat state of deformation by considering only the main section of the soil-structure assembly and assuming that all other sections are equal to this.

The Mohr-Coulomb criterion assumes that the shear strength envelope of the soil is in the form of a line given by:

$$s = c + \sigma.\tan\phi \qquad [1]$$

where "s" is the shear strength, "c" is called cohesion and "φ" is the internal friction angle (Figure 1).

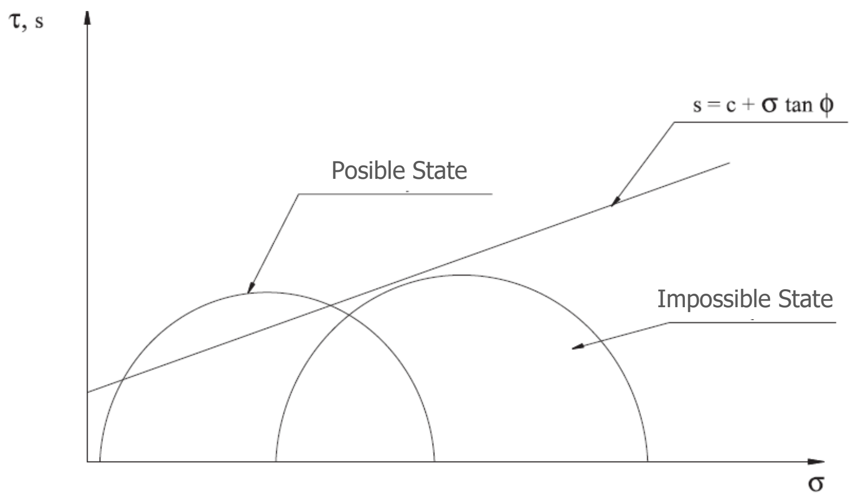


Figure 1 – Mohr – Coulomb Criteria

Thus, cohesion and the friction angle are the parameters of the shear strength of the soil, according to this criterion of rupture, and its determination is fundamental in the determination of thrust. This determination can be made by laboratory tests, such as the direct shear test and the triaxial compression test. They can also be estimated from field trials, or even from other characteristics of the material.

It is important to note that "c" and "φ" are not intrinsic parameters of the soil, but parameters of the model adopted as a criterion of rupture. In addition, the value of these parameters depends on other factors such as moisture content, loading speed and form, and drainage conditions. These values may even vary with time, which leads to the conclusion that the thrust value may also vary with time. This makes the analysis much more complex and it is up to the designer to identify when the conditions of the problem are most unfavorable.

2.1. Non-cohesive soil shear resistance

Non-cohesive soils are represented by sands and gravel, also called granular soils. The shear strength of these soils is mainly due to the friction between the particles that compose them. Thus, the resistance envelope can be expressed by:

$$s = \sigma . \tan \phi \quad [2]$$

On this condition, the cohesion "c" is zero, and the internal friction angle is the only resistance parameter. The main factors that determine the value of the internal friction angle "φ" are:

1. Compactness: is the main factor. The higher the compactness (or the lower the number of voids), the greater the effort required to break the structure of the particles and, consequently, the greater the value of "φ".

2. Granulometry: in the well-graded sands the smaller particles occupy the voids formed by the larger particles, leading to a more stable arrangement with greater resistance. In addition, the coarser sands tend to be naturally more compact because of the weight of each particle. In general, the value of "φ" is slightly higher in coarse sand and gravel.

3. Particle shape: More rounded particles offer less resistance than more irregular particles. Thus, the latter have a larger "φ".

4. Moisture content: soil moisture has a small influence on the resistance of the sands. This is since water acts as a lubricant in the contacts between the particles, decreasing the value of "φ". In addition, when the sand is partially saturated, capillary stresses arise between the particles, which causes the appearance of a small cohesion, called apparent cohesion. However, this cohesion disappears when the soil is saturated or dry.

2.2. Shearing of cohesive soils

The behavior of the clayey soils in the shear is much more complex than that of the granular soils presented in the previous item. This is due to the size of the particles that make up the clays. It is defined as clay the fraction of the soil composed of particles of size less than 0.002 mm. Under these conditions, the specific surface area, defined as the ratio of the total surface area of all particles to the total solids volume, is much higher in the case of clays. This causes physical-chemical surface forces to become predominant in soil behavior. These forces depend greatly on the distance between the particles. Thus, the shear strength increases with the densification, when the particles are approached one another by the effect of a loading. When this charge is withdrawn, the surface forces prevent the particles from returning to the previous situation and then cohesion arises.

The presence of water in the voids of the clay soil also greatly influences its resistance. This is in part due to the fact that the water causes a separation between the particles, reducing the cohesion. On the other hand, in partially saturated clay soils, the effect of suction caused by capillary forces tends to increase cohesion.

Another important feature related to the presence of water, which influences the behavior of clayey soils, is its low permeability. While in the sands any excess pore pressure caused by loading dissipates almost immediately, in the case of clays this dissipation is much slower. Thus, the pore-pressure originated by the loading continues to act, even after the completion of the construction, sometimes for years. There are thus two extreme situations. The situation immediately after the application of the load, when little or no pore-pressure dissipation occurred, called the short-term or non-drained situation and the long-term or drained situation, after the total dissipation of the entire pore-pressure caused by the loading . The soil behavior in each of these two conditions is different, and the project must take this difference into account.

The resistance envelope representing the short-term situation is termed "swift" or "undrained" envelope. This envelope is used in the analysis when it is assumed that no pore-pressure dissipation occurred due to the applied load on the soil. In addition, it is also recognized that the pore-pressure value acting in the field is similar to that acting on the strength tests and therefore need not be determined. In the case of saturated soils, the fast envelope does not present friction:

$$s_u = c_u \qquad [3]$$

Where "cu" is called undrained cohesion. This is because the confident pressure buildup does not translate into increased soil strength since no drainage does not occur in the soil and then the increased confinement is transferred to the water and results in an equal increase in pore pressure.

For partially saturated soils, however, there is an increase in resistance with increasing confinement. This causes the envelope "su" to exhibit a portion of friction. In general it is considered that the situation of complete saturation is more critical and, therefore, this friction is neglected.

At the other extreme, the long-term situation is characterized by the dissipation of all pore-pressure caused by the charge. The resistance envelope representing this situation is called the "s" effective envelope and is used to analyze situations where all the pore-pressure caused by the charge has dissipated. In this case the analysis is done in terms of effective stresses and it is necessary to determine the pore-pressures due to the water table, when present.

In clusters normally densified and saturated, the effective envelope "s'" is not cohesive:

$$s' = \sigma'.\tan \phi' \quad [4]$$

Where "σ’" is the effective normal stress and "φ’" is the effective angle of friction of the ground.

Effective cohesion occurs only in the pre-densified clays, as a result of the overgrazing of the soil. For confining pressures below the pre-compacting pressure, the shear strength is higher than that of the normally thickened clay. As this envelope of a line approaches a range of working stresses which includes stresses below the pre-compacting pressure, the effective envelope becomes:

$$s' = c' + \sigma'.\tan' \phi' \quad [5]$$

Where "c" is effective cohesion.

In the determination of active buoyancy on reinforcement structures, the analysis in terms of effective stresses is usually better, using the envelope of effective resistance of the soil. This is because the active thrust hypothesis characterizes a land unloading, and the long-term situation is generally more unfavorable. Thus, even in the case of masses formed by clayey soils, the effective cohesion is very small, or even null. Thus, it is common to completely disregard cohesion in the calculation of active thrust on reinforcement structures.

3. Water percolation and drainage

The presence of water in the soil influences the behavior of the containment structures in several ways. Firstly, the parameters of soil shear strength, cohesion, decrease as the moisture increases. Also the specific weight of the soil is increased by the presence of water in the voids.

In addition to these influences, the pressure in the water changes the value of the thrust acting on the structure. As an example, see the structure outlined in Figure 2. It is a retaining wall that supports a massive saturated by the effect of intense rains. As the structure is impermeable and at the base of this massif there is a layer also impermeable, there is no water drainage and thus this exerts hydrostatic pressures on the wall. These pressures can, in many cases, overcome the very buoyancy exerted by the soil.

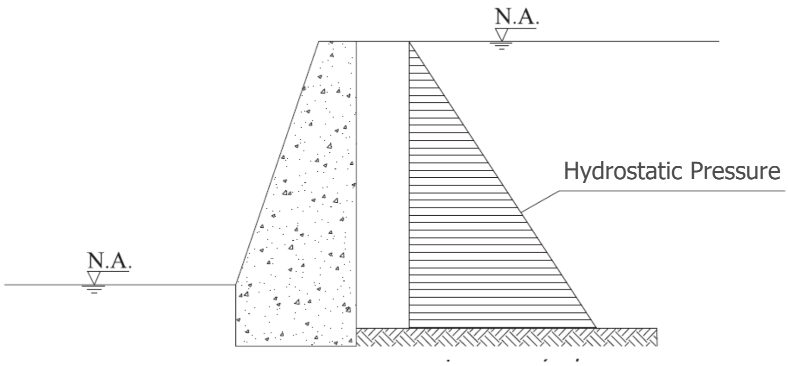


Figure 2 – Wall under hydrostatic pressure

If there is no impermeable layer at the bottom of the solid mass, the water will percolate through the voids of the soil and then the pressure distribution will cease to be hydrostatic. In addition, in the case of reinforcement structures in gabions, the wall itself is permeable and thus the water also percolates through it.

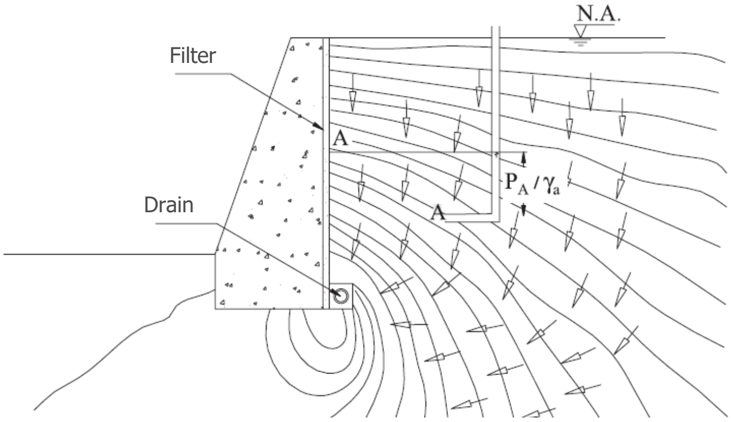


Figure 3 – Retaining wall with drainage system.

The water pressure on the holding structure, in this case, is eliminated as shown in figure 3. In this case there is percolation of water through the soil and the wall. Equipotentials were drawn with the aid of a finite element program. The equipotentials are curves of the same total hydraulic load, which, in turn, results from the sum of the altimetric and piezometric loads. The latter expresses the water pressure inside the soil.

Throughout the soil-structure contact the piezometric load is zero. Inside the massif, however, the water will still be under pressure. In order to determine the piezometric load at any point "A" inside the mass, it is enough to take the equipotential bond that passes through this point and locate the point "A" at the end of this equipotential where the piezometric load is zero. The piezometric load at "A" is given by the difference in height between the points "A" and "A". This is because the total hydraulic charge, which is the sum of the piezometric and altimetric loads, is the same in "A" and "A '".

Although not acting directly on the structure, the pressure of the water inside the mass influences the thrust, increasing its value.

4. Earth Pressure

4.1. Basic Concepts

Earth pressure is the result of the lateral pressures exerted by the soil on a structure of support or foundation. These pressures can be due to the own weight of the ground or to the overloads applied on it.

The value of the thrust on a structure depends fundamentally on the deformation it undergoes under the action of this thrust. It is possible to visualize this interaction by carrying out an experiment using a vertical mobile bulkhead, as shown in figure 4, supporting a soil drop. It is verified that the pressure exerted by the soil on the bulkhead varies with the displacement of the latter

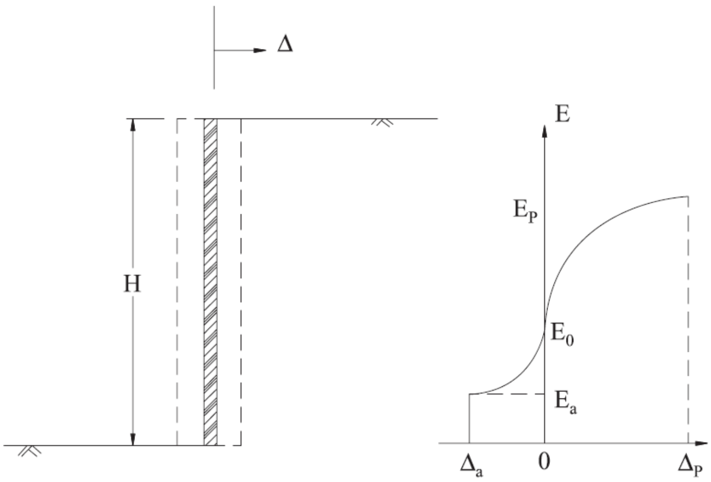


Figure 4 – Thrust over a bulkhead

When the bulkhead moves away from the ground, there is a decrease in thrust to a minimum value that corresponds to the total mobilization of the internal resistance of the ground. This condition is reached even with a small displacement of the bulkhead and is called the active state. The active thrust at this point is then called the active thrust "Ea".

If, on the other hand, the bulkhead is moved against the ground, there will be an increase in thrust to a maximum value where there will again be the total mobilization of the soil resistance. At this maximum value is given the name of passive thrust "Ep", and the deformation condition in which it occurs is called passive state. Unlike the active state, the passive state is only reached after a much greater displacement of the bulkhead.

Should the bulkhead remain motionless in the initial position, the rest thrust "E0" will remain between the values of active thrust and passive thrust. In this condition there is no complete mobilization of soil resistance.

Table 1 shows typical values of the "Δ" displacement of the structure required to achieve the complete mobilization of soil resistance and to reach the active and passive states. It is verified that to reach the passive state is necessary a displacement ten times greater than the necessary one for the active state.

Table 1 – Values of Δ/H

Type of Soil	Values of Δ / H	
	Active	Passive
Compacted Sand	0.001	-0.01
Medium-compacted Sand	0.002	-0.02
Loose sand	0.004	-0.04
Compacted Silt	0.002	-0.02
Compacted Clay	0.01	-0.05

Gravity retaining walls, in general, and flexible ones, if constructed with gabions, allow the deformation of sufficient ground so that its resistance is fully mobilized. Thus, they must be dimensioned under the action of active thrust.

The problem of determining the magnitude and distribution of soil lateral pressure is, however, statically indeterminate and hypotheses are necessary between the relationship between the stresses and soil deformations to reach the solution.

The classical methods used in geotechnics in the determination of active or passive buoyancy adopt a rigid-plastic relationship between tensions and deformations of the soil. This model has the advantage of avoiding the calculation of the displacements of the structure, since any deformation is enough to reach the plastification of the material.

$$K_0 = p_0 / p_v = 1 - \text{sen}\ \phi \qquad [6]$$

Where "p0" is the lateral pressure at rest, "pv" is the vertical acting pressure and "K0" is called the quiescent thrust coefficient at rest. This expression is valid only for normally soils. For pre-compacted soils, the lateral pressure value is higher, depending mainly on the degree of pre-compacting of the material.

4.2. Rankine Theory

When analyzing the tension state of a soil element located at a depth "z" next to the bulkhead of figure 4, one can determine the vertical tension "σ_v" given by:

$$\sigma_v = \gamma.z \qquad [7]$$

Where "γ" is the specific weight of the soil.

While the bulkhead remains at rest, the horizontal stress acting on the element is indeterminate. However, when it is removed from the ground, until the formation of the active state, this tension can be determined from the material resistance envelope, as shown in figure 5.

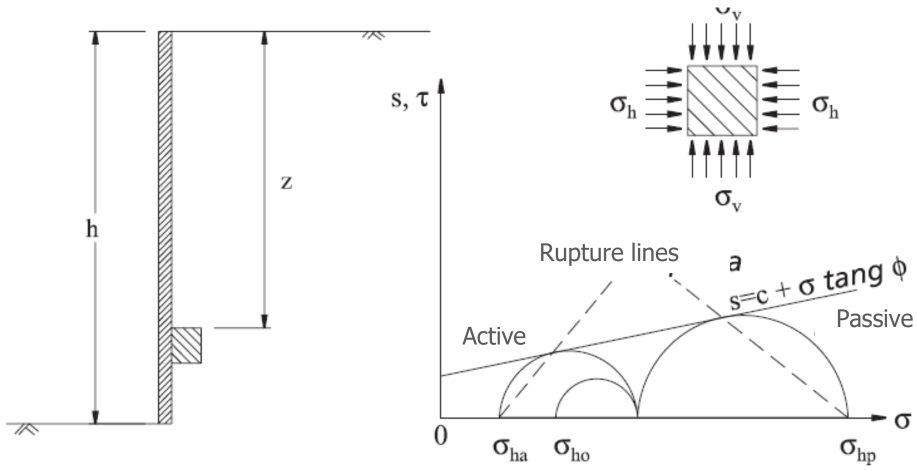


Figure 5 – Determination of lateral pressure

At this time the horizontal voltage "σ_h" is given by:

$$\sigma_h = K_a.\gamma.z - 2.c.\sqrt{K_a} \qquad [8]$$

$$K_a = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right) = \frac{1 - \text{sen}\ \phi}{1 + \text{sen}\ \phi} \qquad [9]$$

Where "Ka" is denominated coefficient of active thrust.

Through this result we can determine the value of the resulting active thrust "Ea" on the bulkhead:

$$E_a = \frac{1}{2}.\gamma.H^2.K_a - 2.c.H\sqrt{K_a} \qquad [10]$$

Where "H" is the total height of the ground clearance.

In case the bulkhead moves to the ground to the passive state, you get:

$$\sigma_h = K_p.\gamma.z + 2.c.\sqrt{K_p} \qquad [\ 11\ ]$$

$$K_p = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) = \frac{1 + \text{sen}\ \phi}{1 - \text{sen}\ \phi} \qquad [\ 12\ ]$$

It is called the passive thrust coefficient, and the resulting thrust "Ep" is given by:

$$E_p = \frac{1}{2}.\ \gamma.H^2.K_p - 2.c.H\sqrt{K_p} \qquad [\ 13\ ]$$

It is verified by these results that the cohesive soil is subject to tensile stresses in its upper portion in the active state. These tensile stresses extend to a depth "z0" given by:

$$z_0 = \frac{2.c}{\gamma} \cdot \frac{1}{\sqrt{K_a}} \qquad [\ 14\ ]$$

It occurs, however, that the ground does not normally withstand tensile stresses. Thus, cracks in the surface are opened up to this depth. Thus, we can not count on these tensions that would diminish the value of the resulting active thrust. In addition, these slots may be filled with water from rainfall, which may further increase the thrust value. The result is the voltage distribution shown in Figure 6. An approximate distribution can be adopted for the purpose of calculation as shown in the same figure and suggested by Bowles.

These tensile stresses do not occur, however, in the passive state, as can be seen in figure 5. Thus, there is no formation of traction slots in the passive state.

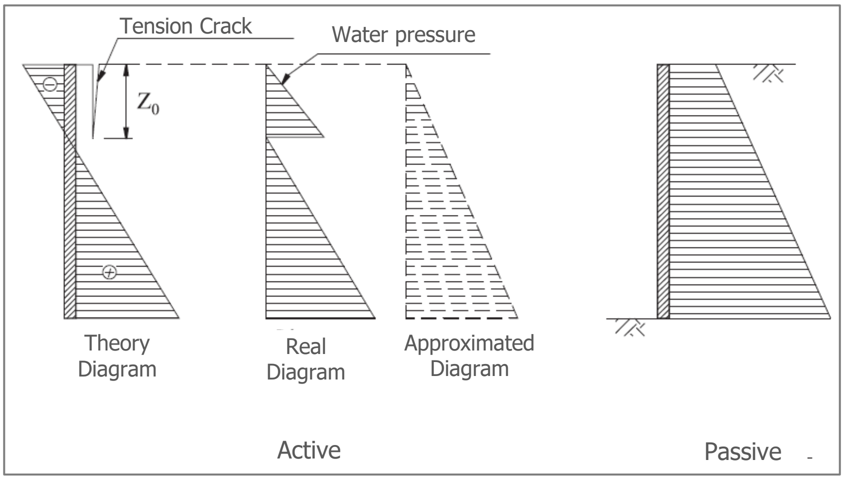


Figure 6 – Earth pressure distribution – Cohesive soil

The directions of the rupture surfaces in the active and passive states are given by the graph of Figure 6 and shown in Figure 7.

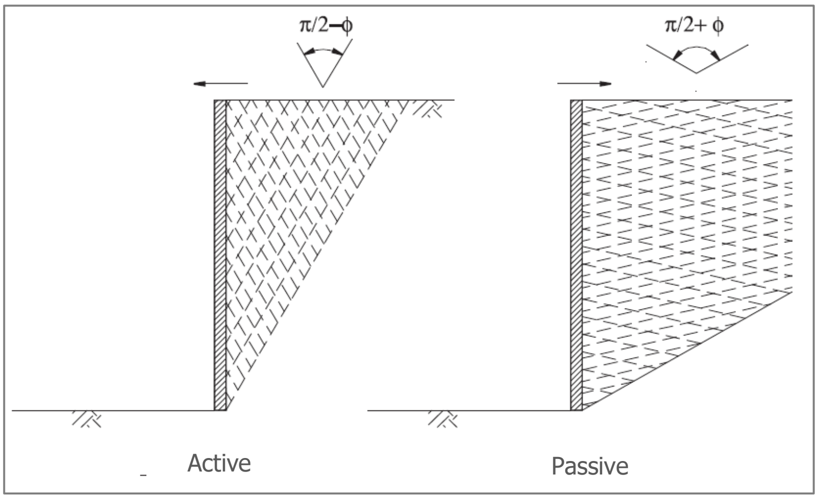


Figure 7 – Rupture plans in active and passive states

If the soil surface is not horizontal, with a slope "i", the vertical pressure value "p_v" will be given by (Figure 8):

$$p_v = \gamma . z . \cos i \qquad [15]$$

As the vertical stress "P_v" has an obliquity "i" in relation to the surface of the soil element shown, it can be decomposed into a normal stress "σ" and a shear stress "τ":

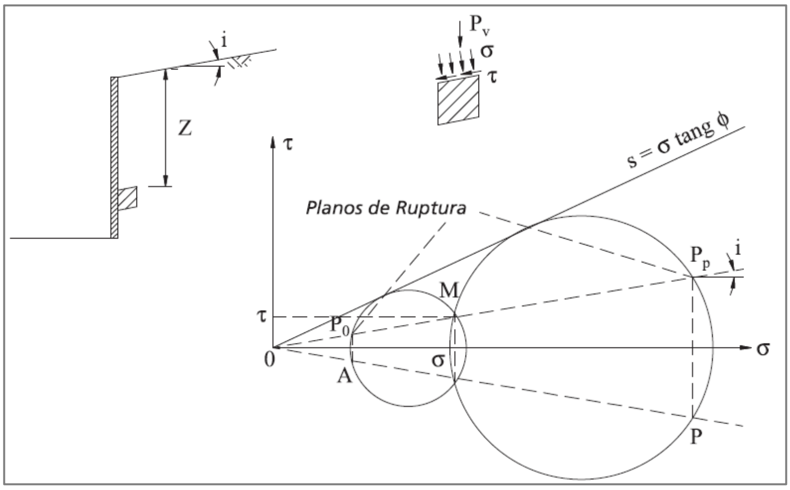


Figure 8 – Determination of earth pressure for i≠0 (c=0)

In Figure 8 are shown the Mohr circles corresponding to the active and passive states, in the case of a non-cohesive soil "c = 0". From this it can be verified that the lateral pressure "pl" on the bulkhead has an obliquity "i" in both states and that the relation between this and the vertical pressure is given by:

$$\frac{p_{la}}{p_v} = \frac{\overline{0A}}{\overline{0M}} = \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} = K_a \qquad [\ 16\ ]$$

For passive case:

$$\frac{p_{lp}}{p_v} = \frac{\overline{0P}}{\overline{0M}} = \frac{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}} = K_p \qquad [\ 17\ ]$$

Therefore, the lateral pressures and the active and passive thrust will be given by:

$$E_a = \frac{1}{2}.\gamma.H^2.K_a.\cos i \qquad [\ 18\ ]$$

$$E_p = \frac{1}{2}.\gamma.H^2.K_p.\cos i \qquad [\ 19\ ]$$

In both cases the thrust direction will be parallel to that of the ground surface.

For the case of cohesive soil, there is no simple analytical expression when the soil surface is not horizontal, it is necessary to determine the lateral pressure graphically with the use of the Mohr circles corresponding to the active and passive states, or by developing the analytical equations correspondents. For this the construction shown in figure 9 is used.

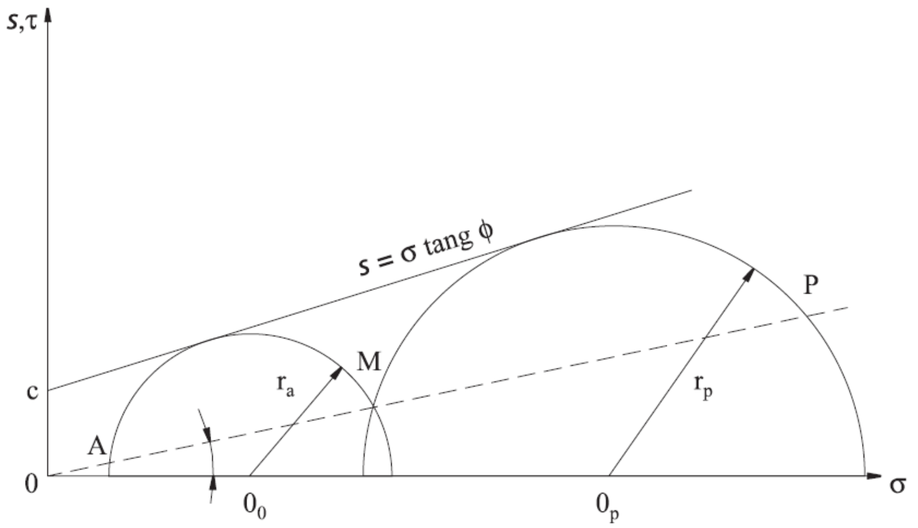


Figure 9 – Determination of horizontal pressures for cohesive soils

Initially determine the point "M" given by:

$$\sigma_M = \gamma.z.\cos^2 i \qquad [20]$$

$$\tau_M = \gamma.z.\text{sen } i.\cos i \qquad [21]$$

The center "0" and the radius "r" of the circle passing through "M" and tangent to the resistance envelope are given by:

$$0 = \frac{\phi_M.\tan^2\phi + c.(\text{sen}^2\phi.\tan\phi) \pm \sqrt{\Delta}}{1 - \cos^2\phi} \qquad [22]$$

$$r = \left(0 + \frac{c}{\tan\phi}\right).\ \text{sen}\,\phi \qquad [23]$$

$$\overline{0M} = p_v = \gamma.z.\cos i \qquad [24]$$

Where the positive signal refers to the passive state and the negative signal, to the active state and:

$$\Delta = 2.c.\sigma_M.\tan^3\sigma.\text{sen}^2\sigma + c^2.\tan^2\sigma.\text{sen}^2\sigma - \tau^2_M.\tan^4\sigma + (\sigma^2_M + \tau^2_M).\text{sen}^2\sigma.\tan^4\sigma \qquad [25]$$

The coordinates of the points "A" and "P" will be given, finally, by:

$$\sigma_A = 0.\cos^2 i - \cos i.\sqrt{(r^2 - 0^2 + 0^2.\cos^2 i)} \qquad [26]$$

$$\tau_A = \sigma_A.\tan i \qquad [27]$$

$$\sigma_p = 0.\cos^2 i - \cos i.\sqrt{(r^2 - 0^2 + 0^2.\cos^2 i)} \quad [28]$$

$$\tau_p = \sigma_p.\tan i \quad [29]$$

$$r = \sqrt{(\sigma_A - 0)^2 + \tau^2_A} \quad [30]$$

$$\tau_A = \sigma_A.\tan \alpha \quad [31]$$

The values of the active and passive side pressures, for depth "z", will be given by:

$$p_{la} = \overline{0A} = \sqrt{\sigma^2_A + \tau^2_A} \quad [32]$$

$$p_{lp} = \overline{0P} = \sqrt{\sigma^2_p + \tau^2_p} \quad [33]$$

Also in this case, tension cracks occur in the active state until the depth "Z0" is given by:

$$z_0 = \frac{2.c}{\gamma} \cdot \frac{1}{\tan\left(\frac{\pi}{4} - \frac{\sigma}{2}\right)} \quad [34]$$

When there is a uniform overload "q" on the mass, its effect on the bulkhead is given by a constant increase of lateral pressure, which will be:

$$p_{la} = (\gamma.z + q).K_a.\cos i \quad [35]$$

$$p_{la} = (\gamma.z + q).K_q.\cos i \quad [36]$$

Therefore the active and passive thrust, in this case, are given by:

$$E_a = \frac{1}{2}.\gamma.H^2.K_a.\cos i + q.H.K_a.\cos i \quad [37]$$

$$E_p = \frac{1}{2}.\gamma.H^2.K_p.\cos i + q.H.K_p.\cos i \quad [38]$$

The point of application of the thrust in all such cases is located in the center of gravity of the lateral pressure diagrams described. Thus, in the case of non-cohesive soil and zero overload, the lateral pressure diagram is triangular, and the point of application of the thrust, both active and passive, is located at a height equal to "H / 3" of the base of the bulkhead.

4.3. Example - Rankine Theory

This example will calculate the Rankine Pressure in a gabion wall with 3 meters (Figure 10).

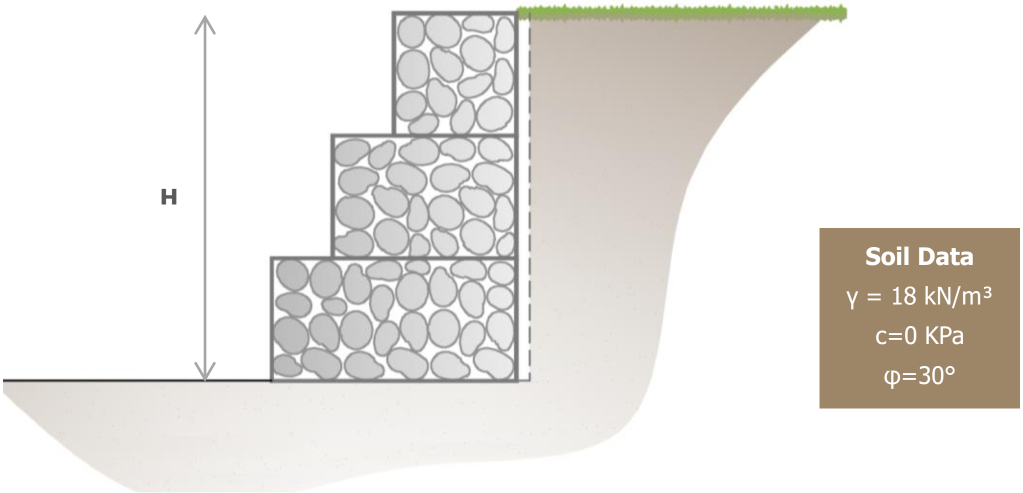


Figure 10 – Example 1 – Rankine Active Thrust

The coefficient of active thrust can be calculated (ka) by:

$$ka = \frac{1 + \sin\varphi}{1 - \sin\varphi} \quad [\,39\,]$$

Then, the active thrust can be calculated:

$$Ea = 0.5\,.\,\gamma\,.\,H^2.\,ka = 0.5\,.\,18\,.\,3^2.\,0.333 = 26.97\ kN/m \quad [\,40\,]$$

The height of application of active thrust will be at the center of gravity of the diagram, on this case, this value will be 1/3 of the Height (Figure 11):

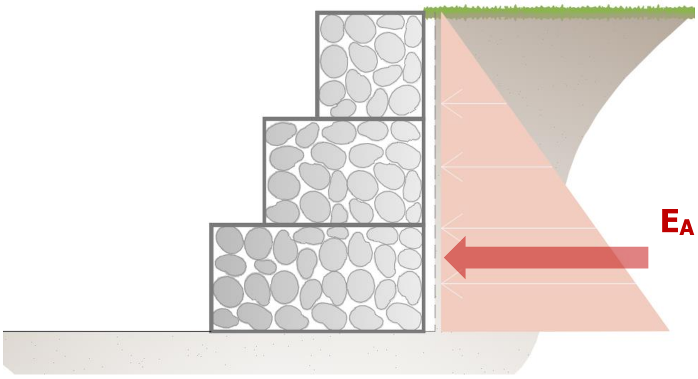


Figure 11 – Point of application of active thrust

The Rankine method can be applied for other cases (Figure 12), like:

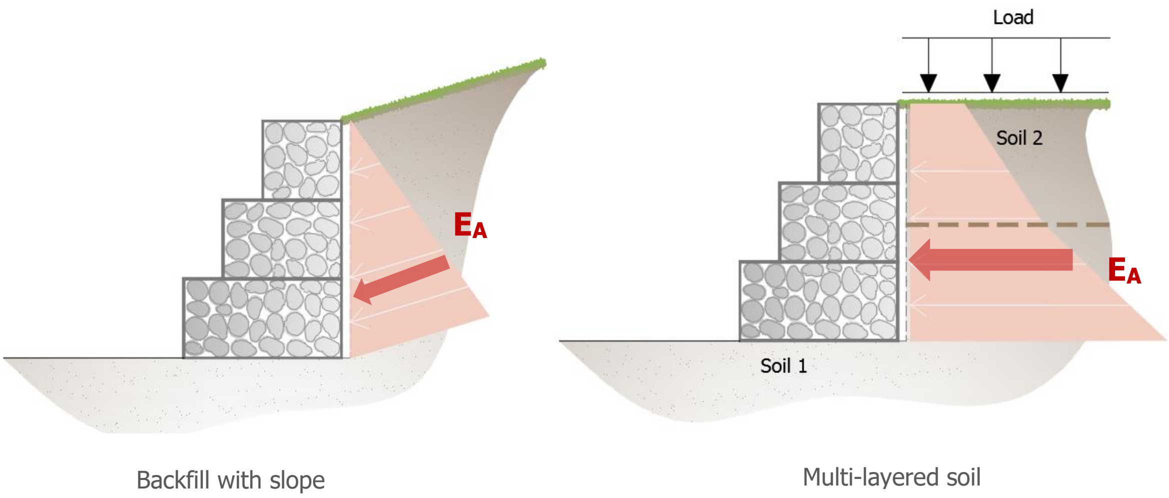


Figure 12 – Examples of Rankine's Theory application

5. Coulomb theory

5.1. Non-cohesive soil

Another way of quantifying the active or passive thrust on a reinforcement structure is to admit that at the moment of total mobilization of the soil resistance, slip or rupture surfaces are formed in the interior of the massif. These surfaces would then delimit a portion of the massif that would move relative to the rest of the ground in the direction of the displacement of the structure. If this portion of the ground is considered as a rigid body, the thrust can then be determined from the equilibrium of the forces acting on this rigid body.

The Coulomb method admits that such rupture surfaces are flat and the thrust is that which acts on the most critical of the flat rupture surfaces.

The advantage of this method lies in the fact that it is possible to consider the occurrence of friction between the reinforcement structure and the soil, in addition to allowing the analysis of non-vertical facing structures.

For the case of non-cohesive soil, the forces acting on the ground wedge formed in the active state are shown in Figure 13. These forces are their own weight "P", the reaction of the "R" mass, which due to the friction angle of the ground has an obliquity "φ" in relation to the rupture surface, and the active thrust "E_A", which also exhibits an obliquity "δ" support This last obliquity is the angle of friction between the soil and the support structure. The potential rupture surface forms an angle "ρ" with the horizontal direction.

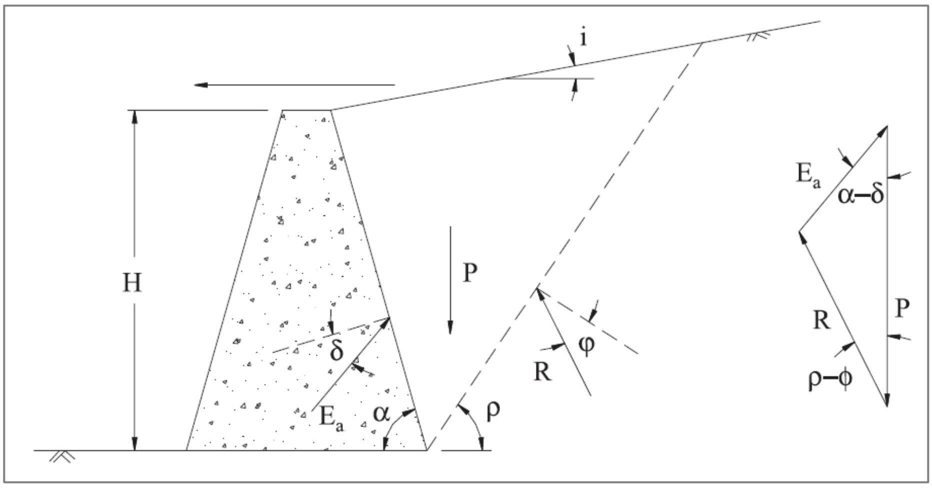


Figure 13 – Forces acting in the wedge – Active pressure

The weight P (Area of surface x unit weight), can be obtained by the formula:

$$P = \frac{\gamma.H^2}{2.\mathrm{sen}^2\,\alpha} \cdot \left[\mathrm{sen}(\alpha+\rho) \cdot \frac{\mathrm{sen}(\alpha+i)}{\mathrm{sen}(\rho-i)}\right] \qquad [\,41\,]$$

The value of Active thrust can be determined by the equilibrium of the forces (Law of sines):

$$\frac{E_a}{\mathrm{sen}(\rho-\phi)} = \frac{P}{\mathrm{sen}(\pi-\alpha-\rho+\phi+\delta)} \qquad [\,42\,]$$

$$E_a = \frac{P.\mathrm{sen}(\rho-\phi)}{\mathrm{sen}(\pi-\alpha-\rho+\phi+\delta)} \qquad [\,43\,]$$

The most critical surface in the active case is the one that takes the value of "Ea" to a maximum, that is, it is obtained from the derivative of the previous expression in relation to the angle of the rupture surface "ρ":

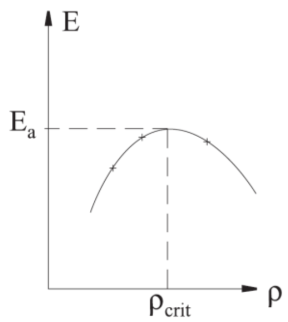


Figure 14 – Maximum active earth pressure

The value of Ea can be defined by:

$$E_a = \frac{1}{2}.\gamma.H^2.K_a \qquad [44]$$

However, for the situation with an infinite slope, the Ea still can be calculation by Ka. This, the "ka" of Coloumb is a simplification made to get the coeficient of horizontal pressure by the formula:

$$K_a = \frac{\mathrm{sen}^2(\alpha + \phi)}{\mathrm{sen}^2\,\alpha.\mathrm{sen}(\alpha - \delta)\cdot\left[1 + \sqrt{\dfrac{\mathrm{sen}(\phi+\delta).\mathrm{sen}(\varphi - i)}{\mathrm{sen}(\alpha-\delta).\mathrm{sen}(\alpha+i)}}\right]^2} \qquad [45]$$

In the passive state there is a reversal in the obliquities of the forces "R" and "Ep" due to the inversion in the direction of the displacement of the structure, and the most critical surface is that which takes "Ep" to a minimum value (figure 15).

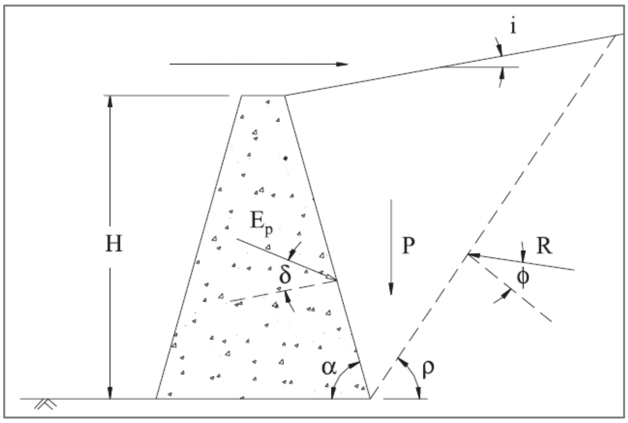


Figure 15 – Forces acting in the wedge – Passive pressure

The value of "Ep" is given by:

$$E_p = \frac{1}{2}.\gamma.H^2.K_p \qquad [46]$$

$$K_p = \frac{\mathrm{sen}^2(\alpha - \phi)}{\mathrm{sen}^2\,\alpha.\mathrm{sen}(\alpha + \delta)\cdot\left[1 - \sqrt{\dfrac{\mathrm{sen}(\phi+\delta).\mathrm{sen}(\phi + i)}{\mathrm{sen}(\alpha+\delta).\mathrm{sen}(\alpha+i)}}\right]^2} \qquad [47]$$

Since in this process there is no determination of the lateral pressure, but the direct determination of the total thrust, it is not possible to determine the point of application of the thrust by the center of gravity of the lateral pressure diagram as in Rankine's theory.

However, the expressions obtained clearly show that the thrust is the result of a triangular distribution of side pressures in both the active and the passive states. Then the point of application of the thrust is located, also in this case, to a height equal to "H / 3" of the base of the structure.

If there is a uniform overload "q" distributed over the mass, this will cause an increase in the thrust value. This increase can be determined by considering the of the overburden that occurs on the ground wedge delimited by the rupture surface (Figure 16). This portion "Q" will add to the weight of the wedge "P" and thus will cause a proportional increase in the other forces acting on the wedge.

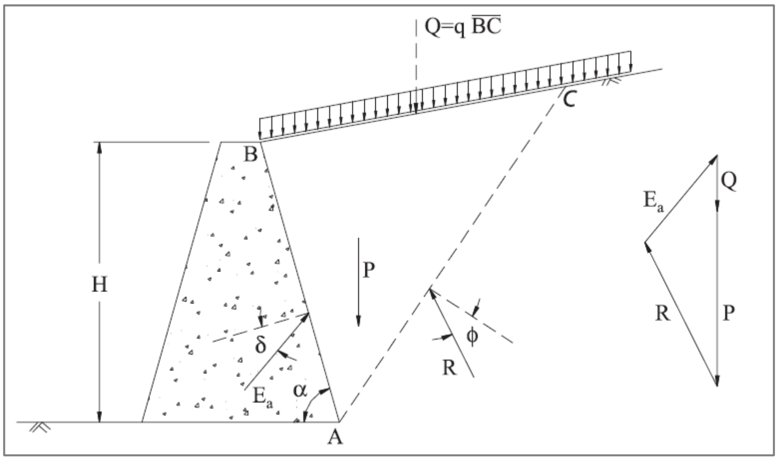


Figure 16 – Thrust due the uniform load

The active thrust is given by:

$$E_a = \frac{1}{2}.\gamma.H^2.K_a.\text{sen } i + q.H.K_a.\frac{\text{sen } \alpha}{\text{sen } (\alpha + i)} \quad [48]$$

From this expression it is noticed that the effect of the overload is evenly distributed along the face, which allows the determination of the point of application of the thrust on the support structure. The first portion of the above expression "0.5.γ.H².ka "is due only to the ground, and therefore is applied to "H/3" from the base of the structure, while the second plot " q.H.Ka. sin(α)/sin(α+i) " is due to overload and will be applied at a height sin (α + i)

equal to "H / 2". The point of application of the total thrust can then be obtained from the center of gravity of the two plots.

5.2. Loads on Backfill

Often overloads occurs on the ground. These overloads come from various sources such as structures built on the bulkhead, vehicular traffic, etc. and cause an increase in thrust.

The simplest case of overloading is the uniform load distributed over the bulkhead (Figure 17). In the analysis by the limit equilibrium method, the portion of the distributed load that is on it must be added to the weight of the soil wedge formed by the rupture surface.

As for the point of application of the resulting thrust, it can be obtained through a parallel to the rupture surface passing through the center of gravity of the soil-overload assembly. Another alternative is to separate the effect of the soil from the effect of the overload and to determine the point of application of each plot through parallels by the centers of gravity of each plot.

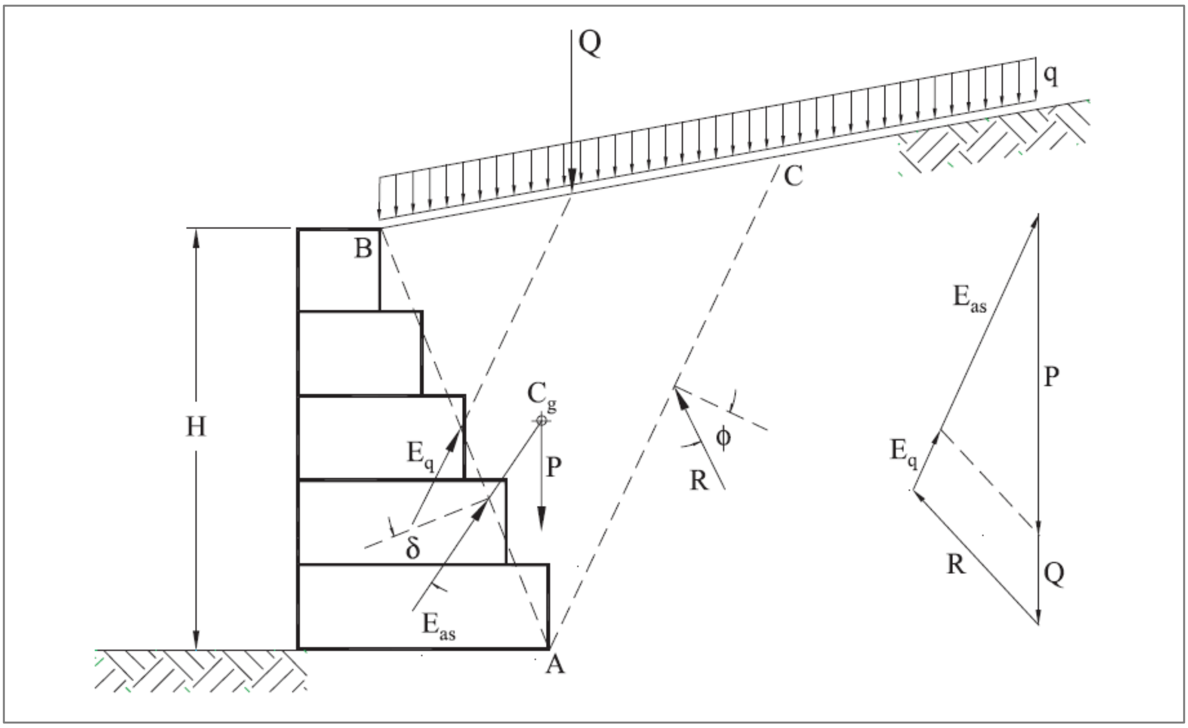


Figure 17 – Uniform load on the wedge

If the conditions of the problem allow the direct use of the Coulomb theory, the effect of uniformly distributed overload can be determined according to the expressions explained early.

Another very common case of overloading is that of load line "Q" parallel to the holding structure as in figure 18.

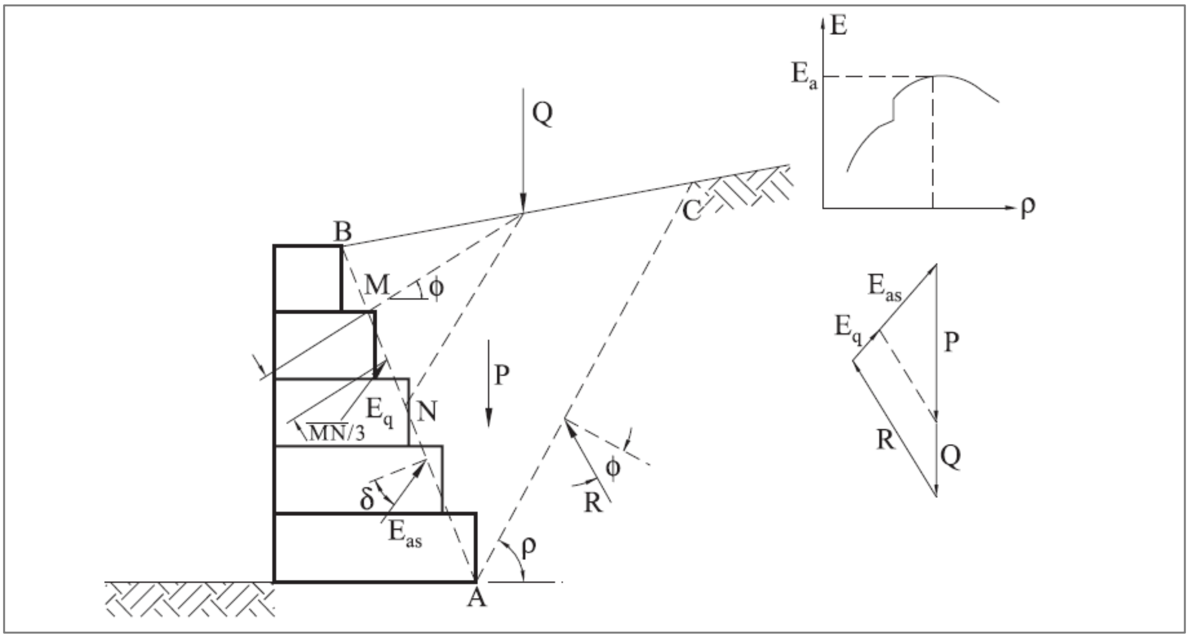


Figure 18 – Line load

In this case, when using the limit equilibrium method, the value of "Q" should be added to the weight of the ground wedge only in the event that the rupture surface ends at a point posterior to the point of application of the load line. Thus, the variation of the thrust with the position of the sliding surface will present a discontinuity at the point corresponding to the position of "Q".

Also in this case, the effects of the "Eas" soil and the effect of the "Eq" load line should be separated from the maximum thrust "Ea". The point of application of the latter is determined according to the theories of Terzaghi & Peck as shown in figure 19.

Another alternative in determining the effect of the load line on the thrust is the use of elastic theory equations obtained by Boussinesq. By this method, the buoyancy due to the ground is determined separately, the presence of the load line being ignored. The effect of the load is simply added to the soil, being determined by the theory of elasticity:

$$\sigma_h = \frac{2.Q}{\pi.H} \cdot \frac{m^2.n}{(m^2.n^2)^2} \qquad [49]$$

where "σh" is the addition of the horizontal pressure due to the load line "Q" and "H", "m" and "n" are indicated in figure 19.

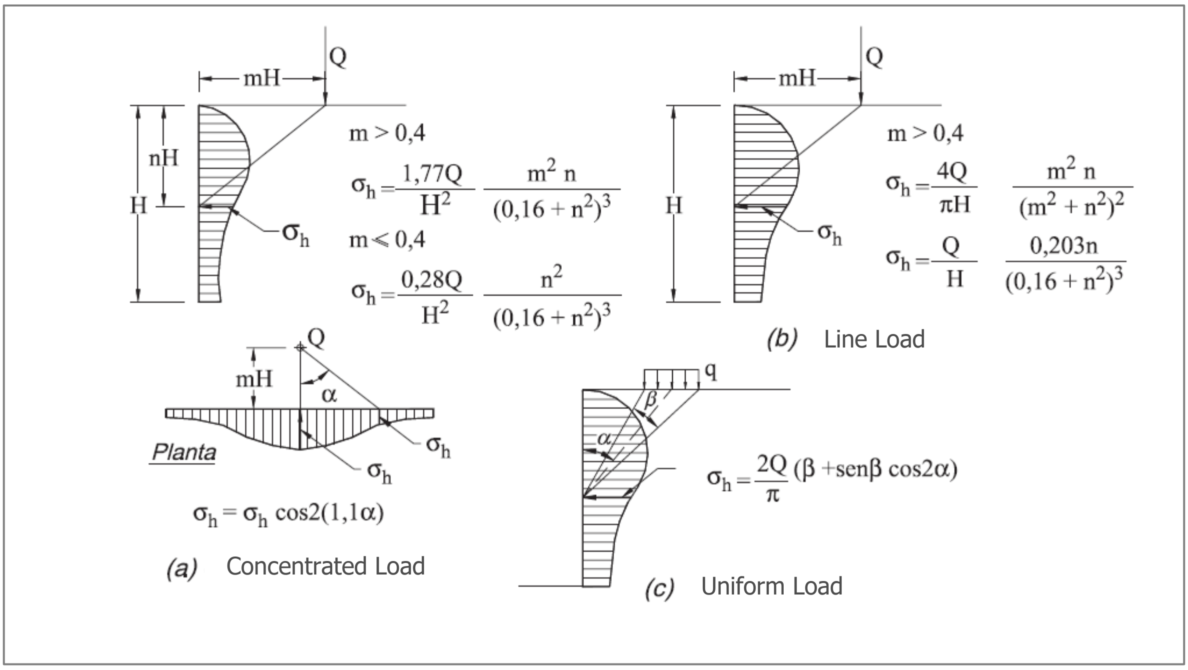


Figure 19 – Effects of surcharge by theory of elasticity

The above expression, however, is only valid for semi-infinite means. As the reinforcement structure has a much greater rigidity than that of the ground, this value must be doubled according to the expressions of figure 19. In this figure are also shown the expressions for the cases of concentrated load and partially distributed load. In all such cases, the expressions shown are increased as explained above.

If the retaining wall is deformable, as is the case of structures constructed in gabions, the value obtained by these expressions can be reduced.

Finally, it should be noted that for the latter method it is assumed that the existence of the overload does not influence the buoyancy due to the ground, that is, the influence of the load in the position of the critical rupture surface is not analyzed.

In fact, it is a superposition of effects that is not at all valid since the effect of the soil is determined by assuming the plastification of the material while the effect of the load is determined assuming a linear elastic model for the material. However, despite these problems, the results obtained by this analysis show good agreement with measurements made in experimental models.

5.3. Cohesive Soil

When the soil that makes up the solid mass is cohesive (c> 0), there is the occurrence of tensile stresses in the upper portion of the massif in the active state, as already seen before.

These stresses cause traction slits to appear which decrease the useful surface area of the rupture surface, thereby increasing the final thrust on the structure.

Thus, the most critical position for the occurrence of a traction slot is at the end of the rupture surface, decreasing its length (Figure 20).

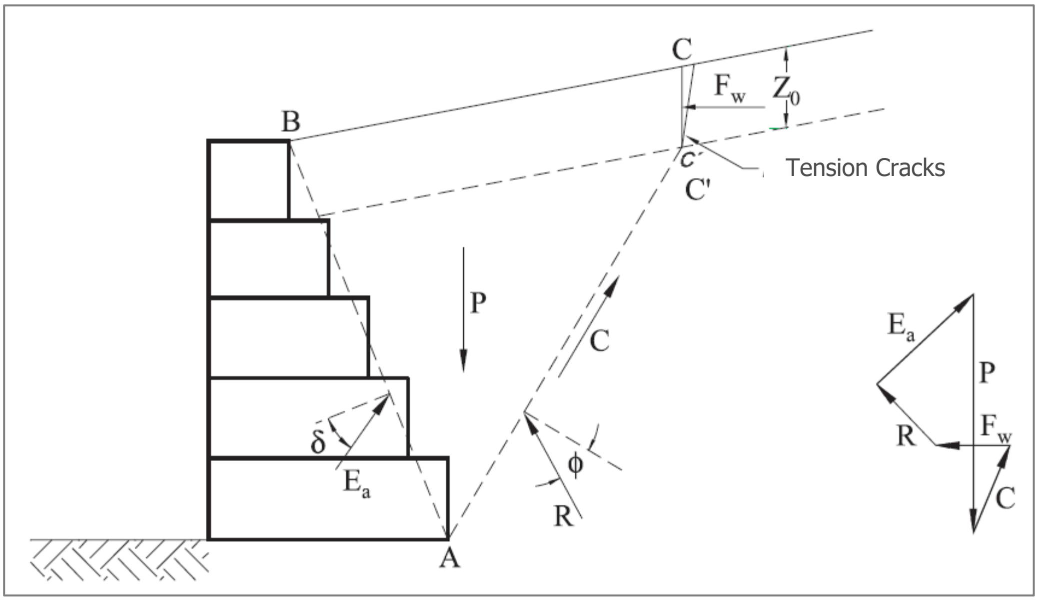


Figure 20 – Active Thrust – Cohesive soils

In addition, as already mentioned, the traction slots may be filled with water from rains, which causes an additional increase in thrust due to the hydrostatic pressure within these traction slits. Thus, forces acting on the ground wedge formed by the rupture surface include this force "Fw" due to the water pressure inside the traction slits, in addition to the strength "C" due to the cohesion of the soil. These forces are determined by:

$$F_w = \frac{1}{2} \cdot \gamma_a \cdot Z_0^2 \qquad [\ 50\ ]$$

$$C = c \cdot \overline{AC'} \qquad [\ 51\ ]$$

where "z0" is the depth of the traction slits, "γa" is the specific weight of the water and "c", the cohesion of the soil.

Using the limit equilibrium method, the thrust is determined from the balance of forces for each hypothetical rupture surface until it is most critical. Each of these surfaces must correspond to a traction slot, since the actual distribution of these slits is random, and the most critical location is that which coincides with the most critical rupture surface.

The point of application of the resulting active thrust "Ea" on the support structure can be adopted as the "H / 3" of the base of the structure. This is justified by the fact that this thrust includes the effect of the water pressure inside the traction slits and by the approximate distribution of lateral pressures presented early.

5.4. Effect of water in active thrust

5.4.1. Partially submerged structure

In regularization works of water courses, it is quite common to construct partially submerged support structures. Figure 21 shows an example.

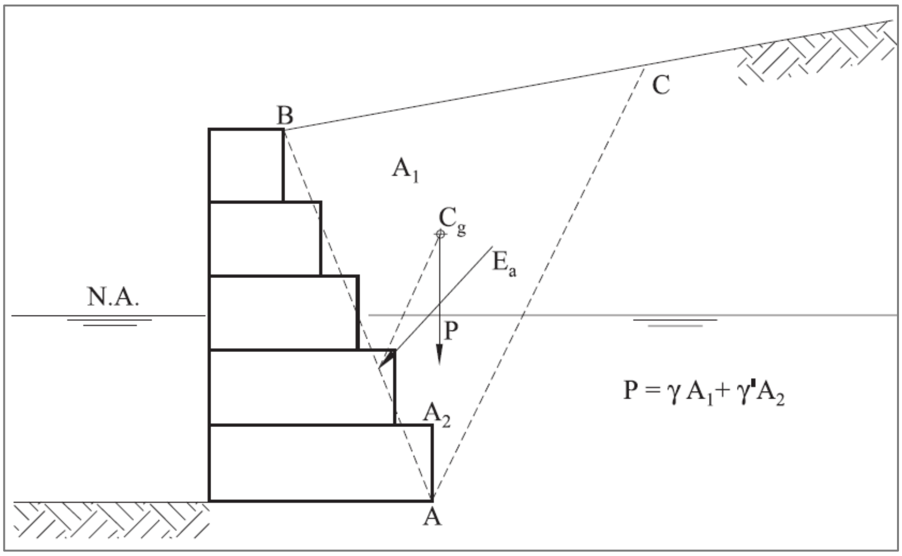


Figure 21 – Retaining wall partially submerged

In these cases, the effect of the existing water in its voids should be separated from the soil effect. This is because soil resistance is due to the pressure between its particles (effective pressure) while the water has no resistance to shear. This type of analysis is known as effective stress analysis.

Thus, to use the limit equilibrium method in this type of structure, it is necessary to determine the balance of forces using the submerged weight of the soil wedge, that is, to calculate the weight of the submerged part of the soil wedge, one should use the specific submerged weight "γ'" of the material.

The thrust "Ea", thus obtained, is then that due only to the weight of the soil particles, it being necessary to add to it the water pressure on the structure. The determination of this pressure is trivial and obeys the laws of hydrostatics. In the specific case of retaining wall of gabions, due to its highly draining nature, stability analysis can be done in terms of effective pressures.

The point of application of thrust "Ea" is determined by a line parallel to the critical rupture surface passing through the center of gravity (of the submerged weight) of the critical wedge.

5.4.2. **Mass under the influence of water percolation**

Another very common case is the occurrence of water percolation through the solid mass. This happens, for example, when the level of the water table that is just below the foundation of the structure rises during the rainy season or, even in structures of the type described in the previous item, there is a sudden reduction of the level of the water course In these cases there is water percolation through the mass in the direction of the reinforcement structure, which increases the value of the thrust on it. For the water not to be trapped behind the wall, further increasing the buoyancy value, self-draining structures, such as gabions, should be used, or the structure of drains and filters that prevent soil particles from being carried.

In order to analyze this type of problem it is necessary to initially determine the flow network formed as shown in figure 22.

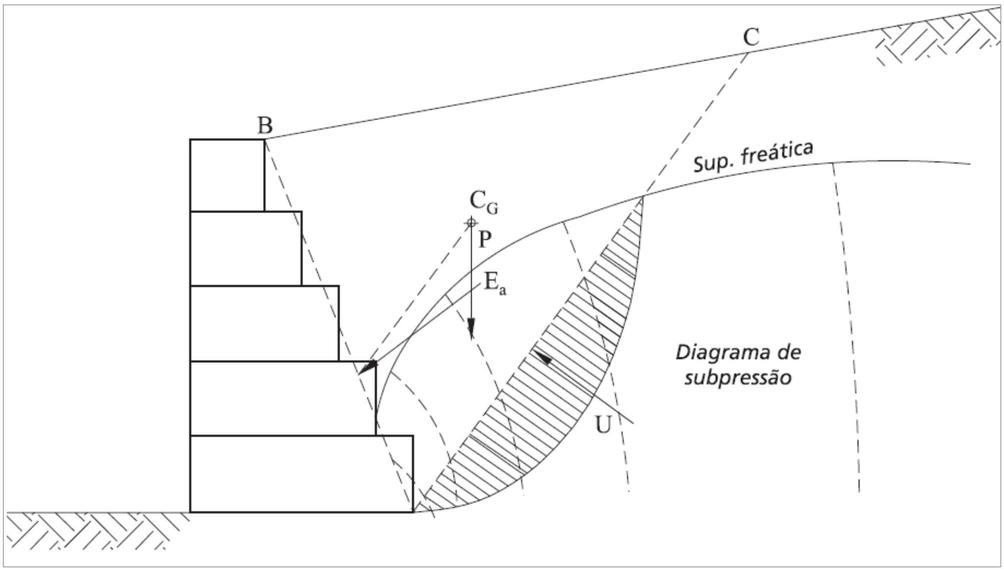


Figure 22 - Water percolation in the backfill

The limit equilibrium analysis can then be performed. The forces acting on the ground wedge formed by the rupture surface include its own weight (determined here using the specific saturated weight "γsat" of the ground) and the "U" force due to the neutral pressure acting on the ground slip surface. The latter is determined from the diagram of subpressions acting on the rupture surface.

A simplified form of force determination "U" consists in the adoption of a "ru" subpressure parameter defined as:

$$r_u = \frac{U}{P}$$ [52]

The point of application of thrust "Ea" can be determined as in the previous item. It should be noted, however, that here thrust "Ea" includes the effect of water. Then the center of gravity of the critical wedge should be determined by its saturated weight.

5.4.3. Non-homogenous soil

If the raised mass is formed by layers of different soils (Figure 23), the limit equilibrium method can still be used.

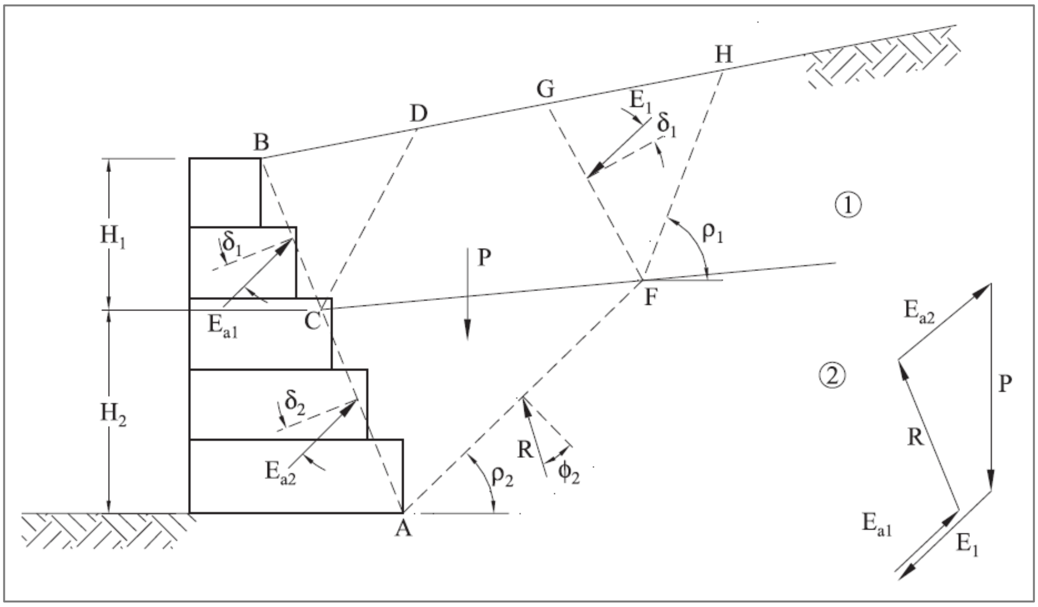


Figure 23 – Non homogeneous soil

First, the thrust "Ea1" caused on the structure by the first soil layer along "BC" is determined using the method already mentioned above.

Next, a rupture surface formed by three planes is considered. The first of these planes starts from the "A" point at the base of the structure (or base of the second soil layer, if there are more than two layers) and extends to the boundary between the second and the first layer "), With a slope" ρ2 "in relation to the horizontal. The second plane departs from this point and proceeds to the surface of the mass (point "G"), in a direction parallel to the inside face of the support structure ("AB"). The third plane starts from the same point ("F") and extends to the surface of the massif (point "H") in an inclined direction of "ρ1" in relation to the horizontal.

Two wedges of soil are formed. One of them with vertices in A, B, G and F, and another with vertices in F, G and H. The effect of the smaller wedge on the larger wedge can then be determined as the thrust "E1", also calculated by the equilibrium limit method , considering an angle of friction between the two wedges equal to the angle of friction "δ1" acting between the upper layer soil and the reinforcement structure.

Known as the "E1" value, the thrust applied by the lower layer can be determined by the equilibrium of the forces acting on the largest ground wedge.

The inclination "ρ2" should then be searched in order to find the most critical position for the bursting surface.

If the number of layers exceeds two, the process must be repeated to include the lower layers until the base of the structure is reached.

The point of application of "Ea1" is determined as already mentioned in the previous items, that is, the "H1 / 3" of the layer base, where "H1" is the thickness of this layer in contact with the surface is flat and there are no overloads. As for the point of application of "Ea2", it can be assumed that the lateral pressure distribution on the support structure is linear and that the rate of variation of this pressure with the height of the structure is:

$$\frac{dp_{l2}}{dh_2} = \gamma_2 . K_{a2} \qquad [\ 53\ ]$$

Where "Ka2" is the active buoyancy coefficient determined by the Coulomb theory. Thus, the lateral pressure at the top and bottom of the second layer can be determined and then the center of gravity of the lateral pressure diagram obtained (Figure 24).

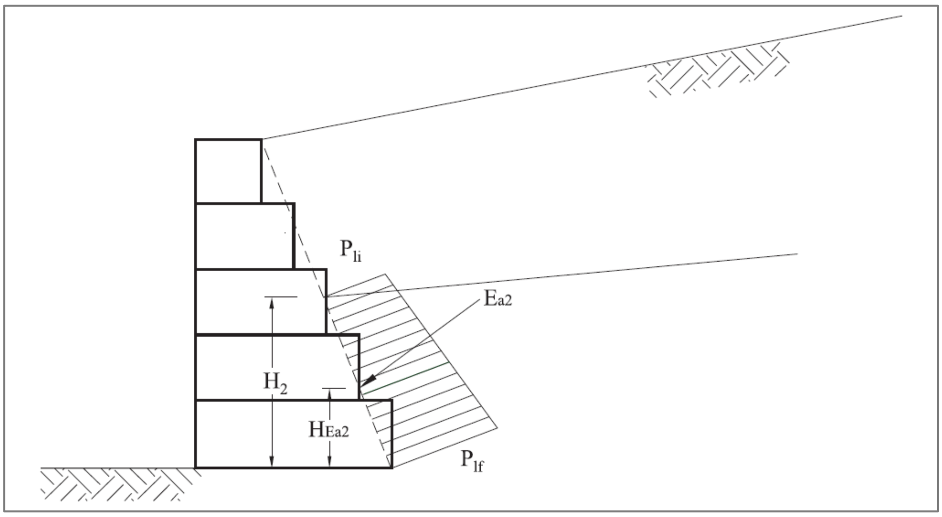


Figure 24 – Earth pressure distribution on the second layer

$$dp_{li} = \frac{E_{a2}}{H_2} - \frac{\gamma_2 . K_{a2} . H_2}{2} \qquad [\ 54\ ]$$

$$dp_{lf} = \frac{E_{a2}}{H_2} + \frac{\gamma_2 . K_{a2} . H_2}{2} \qquad [\ 55\ ]$$

$$H_{E2} = \frac{H_2}{2} - \frac{\gamma_2.K_{a2}.H_2}{H_{a2}} \cdot \frac{H_2^3}{12}$$ [56]

5.4.4. Seismic Effect

During a seismic effect, the active thrust may increase due to horizontal and vertical accelerations of the ground. These accelerations provoke the appearance of inertial forces in the vertical and horizontal directions that must be considered in the balance of forces (Figure 25).

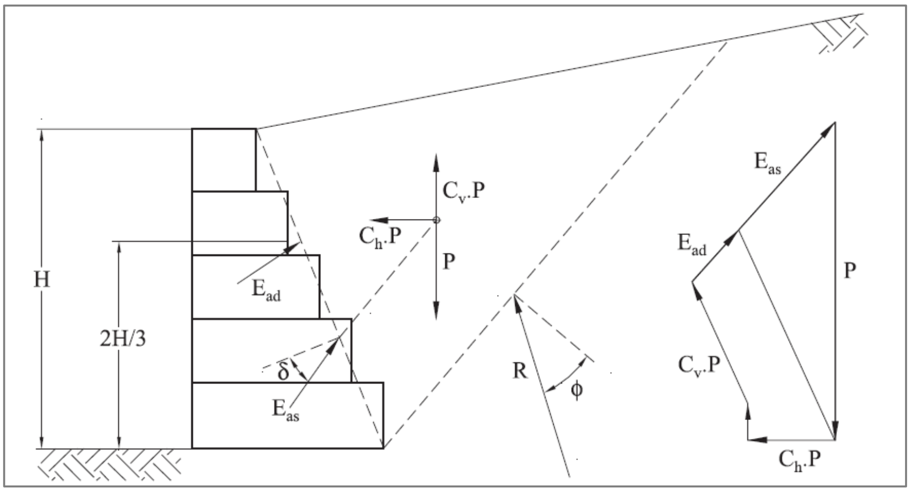


Figure 25 – Forces due the seismic consideration.

These accelerations are usually expressed in relation to the acceleration of gravity "g" and are a function of local seismic risk. Thus, the forces of inertia will be calculated as plots of the wedge weight of soil "P":

$$I_h = C_h \,.\, P$$ [57]

$$I_v = C_v \,.\, P$$ [58]

Where "C_H" and "C_V" are the acceleration ratios in the horizontal and vertical directions.

The acceleration in the horizontal direction has a greater influence on the value of the active thrust and, therefore, is generally the only one considered in the analysis.

The calculated active thrust, then, in this way can be divided into two plots. The first, equal to the static thrust "Eae", has its point of application on the structure determined as in the previous items. The second "Ead" plot is the effect of the earthquake, and its point of application is situated at "2.H / 3" of the base of the structure.

If the mass is submerged, the specific submerged weight "γ" of the ground must be used in calculating the specific weight of the wedge. Therefore, it is also necessary to consider the seismic effect in the mass of water inside the soil.

This mass will cause additional pressure to the static effect, resulting in an additional thrust "U_d" due to water, given by:

$$U_d = \frac{7}{12} . H^2{}_a . \gamma_a . C_h \qquad [\ 59\ ]$$

where "γa" and "Ha" are the specific weight and height of the water respectively. This thrust is applied to "Ha / 3" from the base of the structure.

If the conditions of the problem allow the direct use of the Coulomb theory and, in addition, consider only acceleration in the horizontal direction, the seismic effect can be determined by the expressions mentioned early, correcting the values of the angles "α "And" i "in figure 25.

$$\alpha' = \alpha - \theta \qquad [\ 60\ ]$$

$$i' = i + \theta \qquad [\ 61\ ]$$

$$\theta = \arctan C_h \qquad [\ 62\ ]$$

The thrust "Ea" thus calculated must still be multiplied by "A", which is given by:

$$A = \frac{\operatorname{sen}^2 \alpha'}{\operatorname{sen}^2 \alpha . \cos \theta} \qquad [\ 63\ ]$$

The seismic effect "Ead" will then be given by:

$$A_{ad} = A.E'_a - E_{ae} \qquad [\ 64\ ]$$

Where "Eae" is the static active thrust. The "Ead" difference is applied to "2H / 3" from the base of the wall:

6. Application of theories in Gabion Walls

Gabion walls are structures of gravity and as such can be dimensioned. Thus, the classical Rankine and Coulomb theories, as well as the limit equilibrium method can be used to determine the active buoyancy.

For simpler cases, Coulomb's theory is generally employed in determining active thrust, since it covers a reasonable variety of situations encountered in practice.

The soil characteristics should be carefully evaluated, as the results of the analyzes depend on them. It should be noted that the mass is usually a backfill, preferably executed with non-cohesive material and so it is normal to consider as cohesive soil.
Even when clay soil is used in the backfill, the available cohesion is very small, because in addition to the dents caused by the construction, it should be remembered that the active state is formed in a situation of unloading of the massif; and thus, the most critical situation is that which corresponds to the drained condition of the resistance. Thus, the shear strength envelope most indicated in these cases is the effective envelope (also called drainage envelope), which normally presents a very small or even zero cohesion plot for clayey soils.

For the friction angle "δ" between the ground and the structure, the same value of the angle of internal friction "φ" of the ground can be adopted, since the face of the gabions is quite rough. In the case of a geotextile filter between the upright soil and the wall of gabions, the value of "δ" should be reduced, with a "$\delta = 0.9\varphi$ to 0.95φ" being adopted.

If the specific conditions of the analyzed problem are more complex, not allowing the direct use of the Coulomb theory, the limit equilibrium method is usually used. In this case, however, the work involved in determining the active thrust is considerably larger. That is why computer programs have been developed that help the designer in this task. The Gawac® program distributed by Maccaferri, to the designers, uses the limit equilibrium method to determine active active thrust, which makes it capable of analyzing most of the cases that may arise.

For the calculation of passive thrust, which is the resistance to horizontal displacement offered by the ground in front of the wall, when it is supported at a lower dimension than its surface (Figure 26), Rankine's theory is generally sufficient. One should, however, exercise caution in considering this resistance. Only the passive buoyancy at the front of the wall should be considered when it can be ensured that there will be no excavation or even erosion in the soil situated in front of the holding structure.

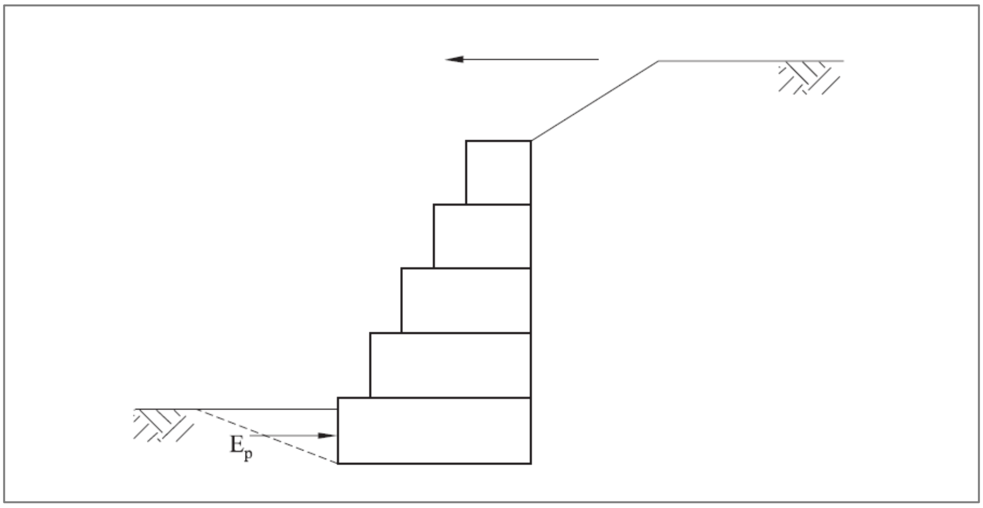


Figure 26 – Determination of passive thrust

The Coulomb and limit equilibrium methods adopt the shape of the rupture surface as being flat, by hypothesis. However, the flat shape does not always lead to the most critical condition for the equilibrium of the ground wedge formed by the rupturing surface. The occurrence of friction along the soil-structure interface ensures that the most critical rupture surface is curved.

More rigorous analyzes, using logarithmic spiral-shaped rupture surfaces, were developed. In the case of active thrust, these analyzes show that the value calculated with the use of flat surfaces differs by no more than about 10%, with this difference being generally within 5% of the calculated value with curved rupture surfaces. This fact justifies the use of flat rupture surfaces for the calculation of active thrust because they are simpler and more comprehensive analysis.

For passive buoyancy, however, the difference between the results obtained by the methods using flat rupture surfaces and those using curved surfaces is much greater. Only when no friction between the soil and the reinforcement structure is considered, the results obtained by the Coulomb theory and by the equilibrium boundary method are correct.

6.1. Gabion wall analysis

It is necessary to check the safety of the hoist structure against the various types of rupture. In the case of gabion retaining walls, the main types of ruptures that may occur are shown in Figure 25.

1. Sliding on the base: occurs when the slip resistance along the base of the wall, added to the passive thrust available in front of the structure, is insufficient to neutralize the effect of active active thrust.

2. Overturning: occurs when the stabilizing moment of the proper weight of the wall in relation to the tipping fulcrum is insufficient to neutralize the moment of the active buoyancy.

3. Foundation bearing capacity: occurs when the pressures applied by the structure on the foundation soil are greater than its load capacity.

4. Global rupture of the massif: slip along a rupture surface surrounding the retaining structure.

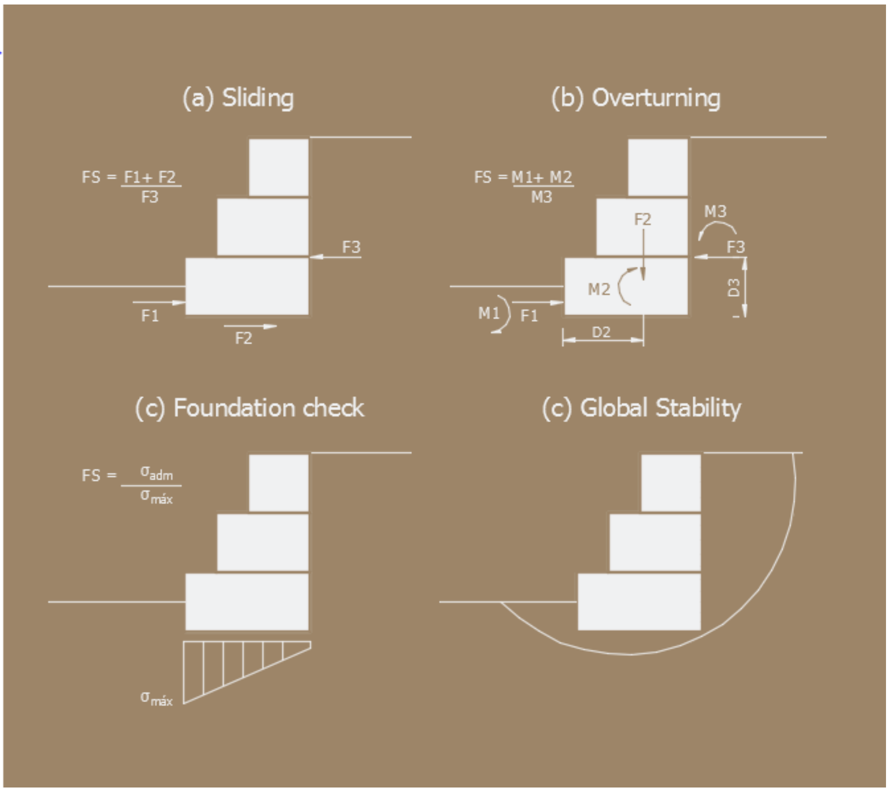


Figure 27 – Type of analysis – Gabion wall

6.1.1. Sliding check

The sliding of the structure occurs when the resistance against sliding along the base of the retaining wall, added to the passive thrust available in front of it, is not enough to counteract the active thrust.

You can define a safety coefficient against sliding:

$$F_d = \frac{T_d + E_{pd}}{E_{ad}} \qquad [65]$$

Where "Ead" and "Epd" are the components of active and passive thrust in the direction of slippage (figure 28).

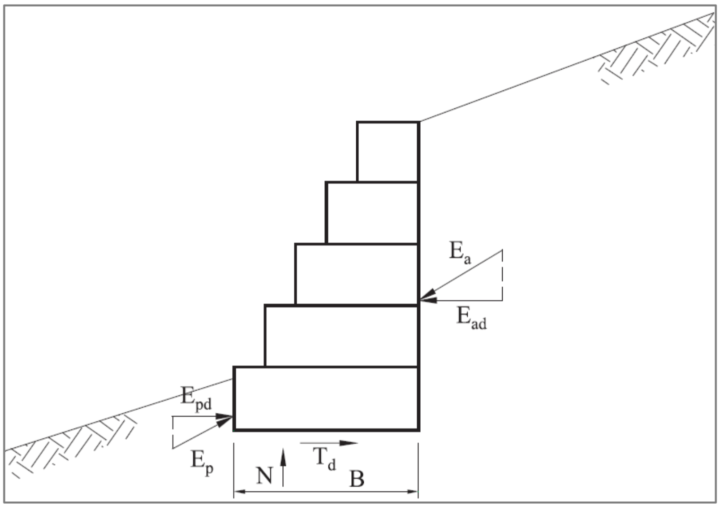


Figure 28 – Sliding Check

The force "Td" is the available resistance along the base of the structure and is:

$$T_d = N.\tan\delta^* + a^*.B \quad [66]$$

where "δ*" is the angle of friction between the foundation soil and the base of the structure, and "a*" is the adhesion between the soil and the base.

The suggested values for "δ*" and "a *" are:

$$\frac{2}{3}\tan\phi \le \tan\delta^* \le \tan\phi \quad [67]$$

$$\frac{1}{3}.\,c \le a* \le \frac{3}{4}.\,c \quad [68]$$

It is also suggested that the value of "Fd ≥ 1.5" should be for non-cohesive soils and "Fd ≥ 2.0" for cohesive soils.

6.1.2. Overturning Check

The overturning of the reinforcement structure can occur when the momentum value of the active thrust in relation to an "A" point located at the foot of the wall (figure 29) exceeds the value of the moment of the proper weight of the structure, added to the moment of the passive thrust. Point "A" is called the fulcrum of tipping. The safety factor against tipping is given by:

$$F_1 = \frac{M_P + M_{E_P}}{M_{E_a}}$$ [69]

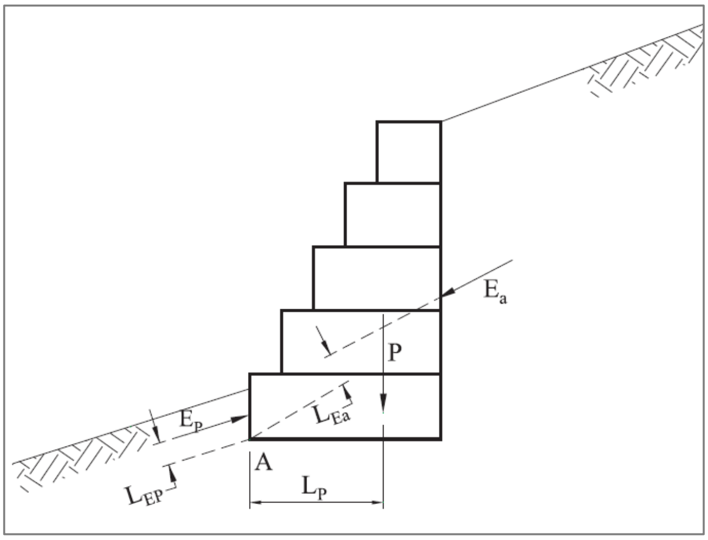


Figure 29 – Overturning check

Another way to define the safety coefficient against tipping is to consider that only the horizontal component of the active thrust "Eah" contributes to the tipping moment, while its vertical component "Eav" contributes to the strong moment. Thus, the security coefficient "Ft" would be:

This last form of "Ft" is more used because it avoids that the safety factor against tipping is negative when the moment of active thrust "MEa" is negative. This situation occurs when the support line of the vector representing the force "Ea" goes down pressures applied by the structure on the foundation soil are greater than its load capacity.

$$F_t = \frac{M_P + M_{E_p} + M_{E_{av}}}{M_{E_a}}$$ [70]

As for the minimum value for the safety factor against tipping, it is suggested that "Ft ≥ 1.5".

6.1.3. Bearing capacity analysis

Another necessary verification is in relation to the pressures that are applied in the foundation by the support structure. These pressures must not exceed the load capacity of the foundation soil.

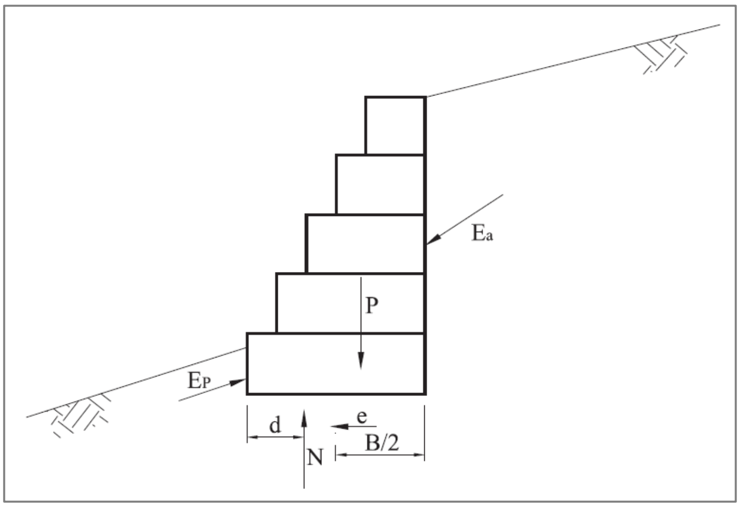


Figure 30 – Point of application – Force N

By means of the balance of moments acting on the reinforcement structure, the point of application of the normal force "N" (figure 30) can be determined:

$$d = \frac{M_P + M_{E_P} + M_{E_{av}}}{N} \quad [71]$$

This normal force is the resultant of the normal pressures acting at the base of the support structure. For these pressures to be determined, the form of their distribution must be known. A linear distribution is usually assumed for these pressures, and then the maximum and minimum values of these will occur at the edges of the base of the structure (Figure 31) and will be given by:

$$\sigma_{máx} = \frac{N}{B} \cdot \left(1 + 6 \cdot \frac{e}{B}\right) \quad [72]$$

$$\sigma_{mín} = \frac{N}{B} \cdot \left(1 - 6 \cdot \frac{e}{B}\right) \quad [73]$$

for "e ≤ B / 6".

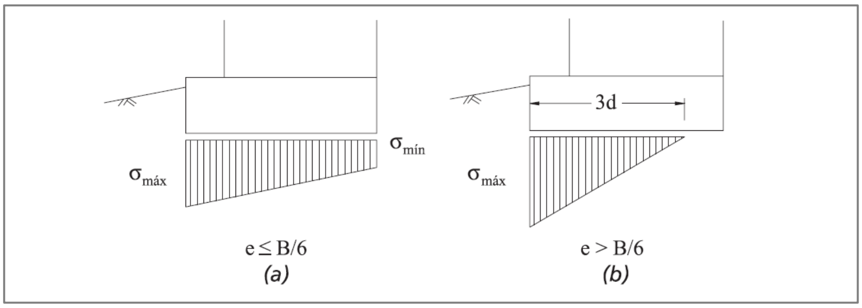


Figure 31 – Distribution of pression on base

If the value of eccentricity "e" is higher than "B/6", there will be a displacement of the internal border, then the type of distribution will be triangular. The maximum pressure will be:

$$\sigma_{máx} = \frac{2.N}{3.d} \qquad [\ 74\ ]$$

This condition must be avoided because of the stress concentration that occurs.

In order to determine the load capacity of the foundation of the wall one can use the expression proposed by Hansen:

$$\sigma_{lim} = c\ .\ N_c\ .\ d_c + q\ .\ N_q\ .\ d_q\ .\ i_q + \frac{1}{2}.\ \gamma\ .\ B\ .\ N_\gamma\ .\ d_\gamma\ .\ i_\gamma \qquad [\ 75\ ]$$

$$q = \gamma\ .\ y \qquad [\ 76\ ]$$

$$i_q = 1 - \frac{T}{2\ .\ N} \qquad [\ 77\ ]$$

$$i_\gamma = i_q^2 \qquad [\ 78\ ]$$

$$d_c = d_q = 1 + 0{,}35\ .\frac{y}{B} \qquad [\ 79\ ]$$

$$d_\gamma = 1 \qquad [\ 80\ ]$$

$$N_q = e^{\pi\ .\ \tan\phi}\ .\ \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \qquad [\ 81\ ]$$

$$N_\gamma = 1{,}8 \cdot (N_q - 1) \cdot \tan \phi \qquad [82]$$

In the above expressions, "γ", "c" and "φ" are the specific weight, cohesion and internal friction angle, respectively, of the foundation soil; "Y" is the height of the ground at the front of the wall relative to the support dimension, and "T" is the tangential force acting at the base.

The maximum permissible pressure shall be:

$$\sigma_{adm} = \frac{\sigma_{lim}}{3} \qquad [83]$$

6.1.4. Global Stability

In addition to the rupture forms mentioned in the previous items, global rupture of the massif may occur along a rupture surface that surrounds the retaining structure without touching it. This type of rupture occurs mainly when there are layers or zones of less resistant soils below the foundation of the retaining wall.

This form of failure is similar to that occurring on slopes, and therefore, the methods used in slope stability analysis can also be used here. The most commonly used slope stability analysis methods are those that analyze the part of the sliding mass as rigid blocks and the methods that analyze it as a single block divided into slices.

The methods of the first type generally use flat rupture surfaces (Figure 32) as the wedge method, whereas those of the second type generally use cylindrical rupture surfaces such as the Fellenius method and the Bishop method (figure 32).

The wedge method considers that the rupture surface is formed by a series of planes delimiting rigid wedges. The equilibrium of these rigid wedges requires that a portion of the resistance be mobilized along these planes. The ratio between the available resistance along the rupture surface and the mobilized resistance is the safety coefficient against the rupture of the mass. The most critical surface is then determined by a process of attempts that seeks to identify the one that presents the lowest value for the safety coefficient.

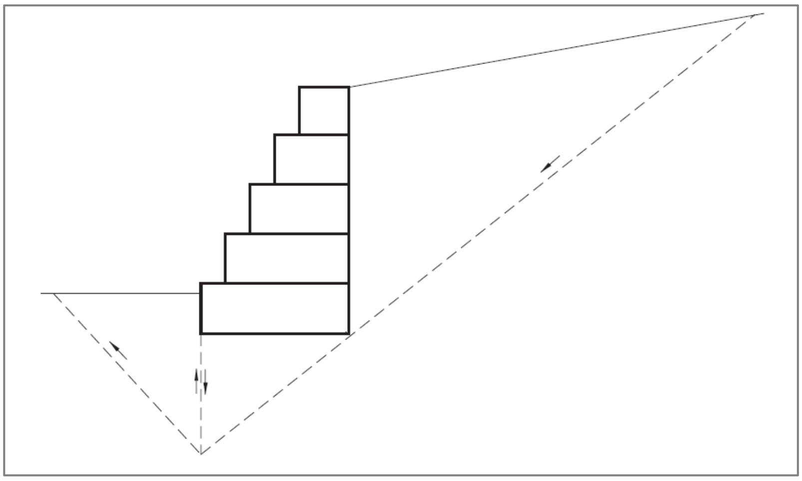


Figure 32 – Global stability analysis – wedge method

The analysis described above is quite similar to that made in the check against the slip of the structure along the base (Sliding Check). There also the planes of rupture form three rigid "wedges": the active wedge, the support structure and the passive wedge (figure 33). The main difference is that the equilibrium of the active wedge is considered to be the total mobilization of the shear strength along the AB and AC surfaces. This means to consider a unit safety coefficient value for sliding along these surfaces. Thus, the slip coefficient "Fd" is restricted to the surfaces of the base of the wall and the passive wedge. Since all the available resistance along the surfaces of the active wedge has been mobilized, the resistance required to balance the assembly along the surfaces where the calculated "Fd" is smaller results in a numerically higher value for the latter in relation to the safety coefficient against global rupture.

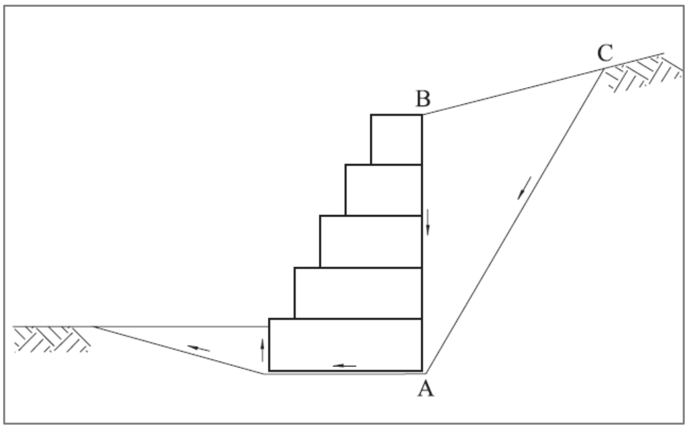


Figure 33 – Wedges considered in sliding analysis

This superiority, however, does not mean greater security, but is only a result of the method of calculation. Thus, the minimum values required for an analysis against overall rupture should also be lower than those required against sliding along the base.

As for the methods that use cylindrical surfaces, their form of determination of the safety coefficient is equivalent to the one of the method of the wedges, since also consider the partial mobilization of the resistance along the whole surface of rupture. They are thus subject to the same observation made above.

The great advantage of the methods that subdivide the potentially unstable material into slices is the possibility of considering many different situations such as layers of different soils, neutral pressures, water table, overload, etc. In addition, the consideration of cylindrical rupture surface is more realistic because it is better approximated of the ruptures observed. For this reason, they are widely used in the stability analysis, both of slopes and of retaining walls. Among these methods, the simplest Bishop method, described below (Figure 34), is the most used.

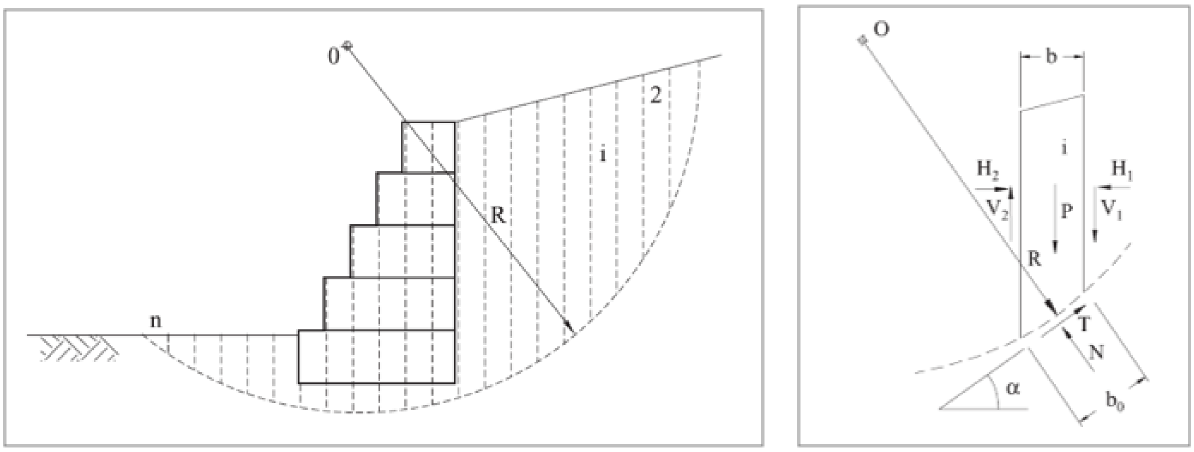


Figure 34 – (a) Bishop slice method (b) forces acting on slice

First, an arbitrary cylindrical rupture surface is allowed, and the material delimited by this surface is divided into slices (Figure 34a). The forces acting on each of these slices are shown in Figure 34b. These are the lamellar weight, the normal "N" and tangential "T" forces acting on the rupture surface and the horizontal "H1" and "H2" and vertical "V1" and "V2" forces acting on the side faces of the coverslip.

By making the balance of forces in the vertical direction we obtain:

$$N.\cos\alpha = P - T\,.\,\text{sen}\,\alpha - (V_1 - V_2) \qquad [84]$$

The tangential force "T" is given by:

$$T = \frac{s\,.\,b_0}{F} = \frac{s\,.\,b}{F.\cos\alpha} \qquad [85]$$

where "F" is the safety coefficient (allowed for all slices) against rupture, and "s" is the shear strength on the coverslip, given by:

$$s = c + \sigma \, . \tan\phi = c + \frac{N.\cos\alpha}{b} . \tan\phi \qquad [86]$$

It can be assumed that "V1 - V2 = 0" with small loss of precision in the result. Thus:

$$N = \frac{P}{\cos\alpha} - \frac{s.b}{F \, . \cos\alpha} . \tan\alpha \qquad [87]$$

Then, the resistance "s" will be:

$$s = \frac{c + \frac{P}{b} . \tan\phi}{1 + \frac{\tan\alpha \, . \tan\phi}{F}} \qquad [88]$$

Making the global balance of moments in relation to the center of the rupture arc and remembering that the sum of the moments of the lateral forces between the slices is zero:

$$\sum_{i=1}^{n} (RT_i) = \sum_{i=1}^{n} (R.T_i \, .\text{sen}\,\alpha_i) \qquad [89]$$

$$R \, . \sum_{i=1}^{n} \frac{s \, . \, b}{F.\cos\alpha} = R \, . \sum_{i=1}^{n} (P \, .\text{sen}\,\alpha) \qquad [90]$$

$$F = \frac{\Sigma\,(s\,.b \,/\, \cos\alpha)}{\Sigma\,(P \, . \, \text{sen}\,\alpha)} \qquad [91]$$

Finally:

$$F = \frac{\Sigma \dfrac{\dfrac{(s\,.b + P.\tan\phi)}{\cos\alpha + \tan\phi.\text{sen}\,\alpha}}{F}}{\Sigma\,(P \, . \, \text{sen}\,\alpha)} \qquad [92]$$

As the security coefficient "F" appears on both sides of the expression, its determination is iterative.

It is necessary to search several surfaces of rupture until finding the most critical (smaller value of "F"). As for the identification of a rupture surface three parameters are required (horizontal and vertical position of the center "O", besides the value of radius "R"), this research is very laborious and there are several search algorithms that can be used. One of the most efficient of them uses a modified version of the Simplex method, which is usually employed in operational research.

The Gawac 3.0G program performs this type of analysis by the Bishop method and employs the Simplex algorithm to determine the most critical rupture surface.

6.1.5. Internal Stability

In addition to the previous stability checks, the internal stability of the retaining wall must be checked. The retaining walls may be subjected to excessive internal stresses caused by the external loading of the thrust and overloads. Thus, this check is made specifically for each type of retaining wall.

In the case of gabion walls, the safety against sliding of the upper and lower gabion blocks must be verified. For each gabion block level, the slip analysis is performed considering the total height of the structure from the top to that level for the active thrust calculation and considering the friction between the blocks as the resistance along the base. The maximum pressure on each layer is given by:

$$\sigma_{\text{máx}} = \frac{N}{2d} \qquad [93]$$

Then, the maximum pressure is compared with the allowable pressure of the gabions, which is given, empirical by:

$$\sigma_{\text{adm}} = 50 \, . \gamma_G - 30$$

Where γ_G is the unit weight of the gabion, given in tf/m³.

There’s also the calculation of the shear force on each layer, which is active thrust force component parallel to each gabion layer). The allowable shear resistance is:

$$\tau_{\text{adm}} = \text{N}.\tan(\varphi) + cg$$

Where "φ" is the gabion friction angle, it is assumed 45°, and "cg" is the gabion cohesion, which can be taken, empirical by the Mesh weight (Pu):

$$c_g = 0.30 \,.\, Pu \,.\, 0.50$$

6.2. Calculation Scheme

For determining the surface area of application of active thrust, there are two cases to consider. In the first of these cases, the geometry of the gabions is such that the face in contact with the solid mass is flat, as shown in Figure 35 (a). In this case, the plane of application of the active thrust is clearly defined by this face.

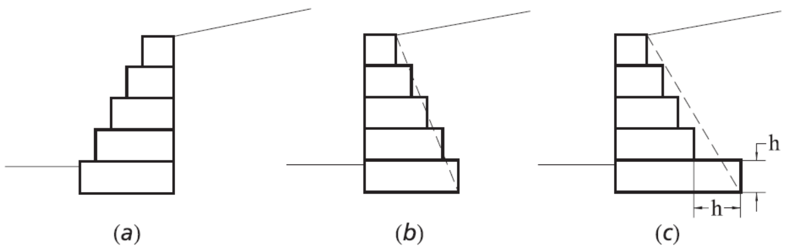


Figure 35 – Application surface of active thrust

In the other case, shown in Figure 35 (b), the gabions are arranged to form steps on the face in contact with the mass. In this case it is necessary to establish a plan of application of the fictitious thrust as shown in the same figure. If the base gabion layer extends into the mass, as shown in Figure 36 (c), a point on the underside of the distal gabion base shall be used as the lower end of the thrust application surface of the projection "h" of the gabions layer immediately above. The part of the base located beyond this point will be considered as an "anchoring" of the wall in the massif.

6.2.1. Choosing the soil parameters

For the determination of the active thrust that acts on the reinforcement structure, it is necessary that the parameters of the ground upright are correctly selected. These parameters are their specific weight "γ", theinternal friction angle "φ" and their cohesion "c". Specific weight can be determined from in situ tests.

The value of the angle of internal friction of the ground must be determined from shear strength tests such as direct shearing or triaxial compression. Preferably the analysis should be made based on the actual normal stresses acting on the mass. Thus, tests must be carried out to determine the effective resistance envelope of the soil.

As for the cohesion of the soil, it is generally taken as null "c = 0". This is because the solid mass is a backfill and in this case the value of effective cohesion is very small, even for clay soils.

In any case, the use of materials with a high clay content in the embankment should be avoided. These soils present several problems. Firstly, they make drainage difficult, since they have low permeability. In addition, they are often expansive when there is an increase in moisture, which causes an increase in buoyancy.

In addition to these parameters it is also necessary to establish the value of the friction angle "δ" between the ground and the structure along the surface of the active thrust. This value can be taken as equal to the internal friction angle of the soil "δ = φ". This is because the surface of the gabions is quite rough, which allows a firm contact between the ground and the structure. If, however, a geotextile filter is used between the face of the wall and the solid mass, the value of this friction angle should be reduced to "δ = 0.9 to 0.95 φ".

6.2.2. Calculation by Coulomb's Theory

The active thrust that acts on the structure can be determined directly by the expressions of the Coulomb theory shown in item 36, when:

- The soil is homogeneous;
- The upper surface of the solid mass is flat;
- The soil is non-cohesive;
- The water table is below the base of the wall;
- There are no irregular overloads on the bulkhead.

If these conditions are satisfied, the active thrust will be given by:

$$E_a = \frac{1}{2} . \gamma . H^2 . K_a \quad [\,94\,]$$

$$K_a = \frac{\text{sen}^2(\alpha + \phi)}{\text{sen}^2\alpha . \text{sen}(\alpha - \delta) . \left[1 + \sqrt{\dfrac{\text{sen}(\phi + \delta) . \text{sen}(\phi - i)}{\text{sen}(\alpha - \delta) . \text{sen}(\alpha + i)}}\right]^2} \quad [\,95\,]$$

and "H", "α" and "i" are shown in figure 36.

The value of "Ka" can also be obtained directly from abacuses.

If there is a uniform overload "q" distributed over the mass, the active thrust value will be:

$$E_a = \frac{1}{2}.\gamma.H^2.K_a + q.H.K_a.\frac{\text{sen }\alpha}{\text{sen}(\alpha + i)} \quad [96]$$

$$q = q \cdot \frac{\text{sen }\alpha}{\text{sen }(\alpha + i)} \quad [97]$$

The point of application of active thrust will be:

$$K_{E_a} = \frac{\gamma.H^2 + 3.q.H}{3.\gamma.H + 6.q} \quad [98]$$

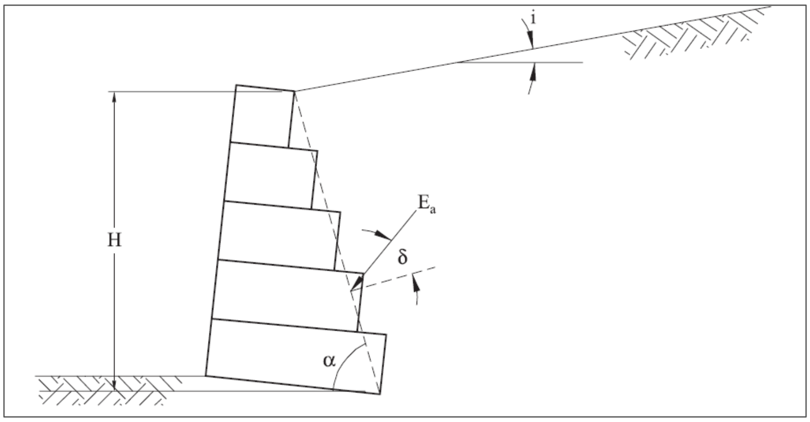


Figure 36 – Active Thrust - Coulomb

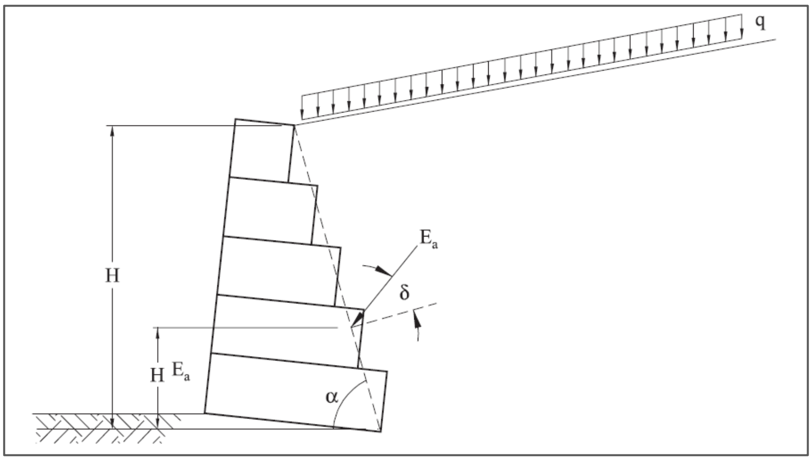


Figure 37 – Point of application – Active Thrust

If the seismic effect is to be considered by means of a horizontal acceleration coefficient "Ch", this seismic effect can be determined by correcting the values of the "α" and "i" angles of figure 36.

$$i' = i + \theta$$ [99]

$$\alpha' = \alpha + \theta$$ [100]

$$\theta = \arctan C_h$$ [101]

The thrust "EA", thus calculated, must still be multiplied by "A", given by:

$$A = \frac{\text{sen}^2\,\alpha'}{\text{sen}^2.\alpha.\cos\theta}$$ [102]

The seismic effect "E'ad" will then be given by:

$$E_{ad} = A.E'_a - E_{ae}$$ [103]

Where "E'ae" is the static active thrust. The difference "E'ad" is applied to 2/3H of the base of the wall.

6.2.3. Equilibrium Limit Method Calculation

Surface of the irregular mass: When the outer surface of the solid mass is not flat, as shown in figure 38, it is necessary to use the limit equilibrium method in determining the active thrust:

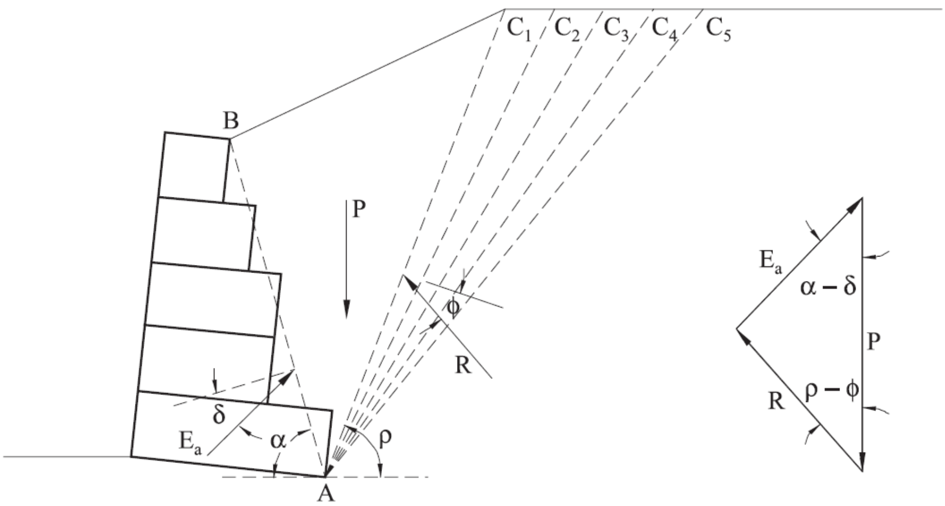


Figure 38 – Irregular mass

Initially, some flat hypothetical rupture surfaces are drawn from the "A" point. Each of these surfaces will define a wedge of rupture. For each of these wedges the weight "P = γ.a" is determined, where "a" is the area of the wedge. The slope "ρ" of the rupture surface is also determined for each wedge.

The value of the active thrust "Ea" is then determined for each of the wedges by the balance of the forces acting on it:

$$E_a = P.\frac{\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\alpha + \rho - \phi - \delta)} \quad [\ 104\ ]$$

With these values of "Ea" a graph is then constructed as in figure 39, interpolating a curve connecting the obtained points.

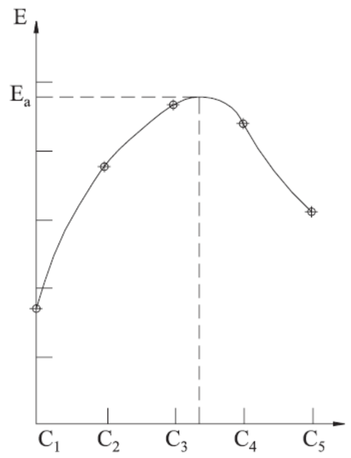


Figure 39 – Active Thrust values due the inclination ρ

The maximum point of the "Ea" variation curve then determines the value of the active thrust acting on the structure and position of the critical rupture surface.

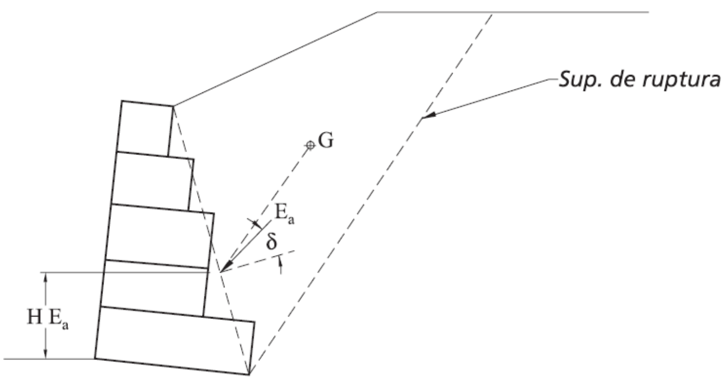


Figure 40 – Application point of Active Thrust

In order to define the point of application of the active thrust, the center of gravity "G" of the ground wedge formed by the critical rupture surface is determined and a parallel is drawn parallel to it by this point, as shown in figure 3.7.6 . The point of application of "Ea" will be at the intersection of this parallel with the application surface of the thrust.

Distributed loads: If, in addition to the irregular surface, there are overloads distributed over the solid mass, the limit equilibrium method is used in the same way as in the previous item, only adding to the weight of each of the total value of the load applied to it.

Thus, as shown in figure 41, the load "Q" to be added to the "P" weight of the wedge was divided into two "Q1" and "Q2" plots, each resulting from the multiplication of the load distributed by the respective distribution.

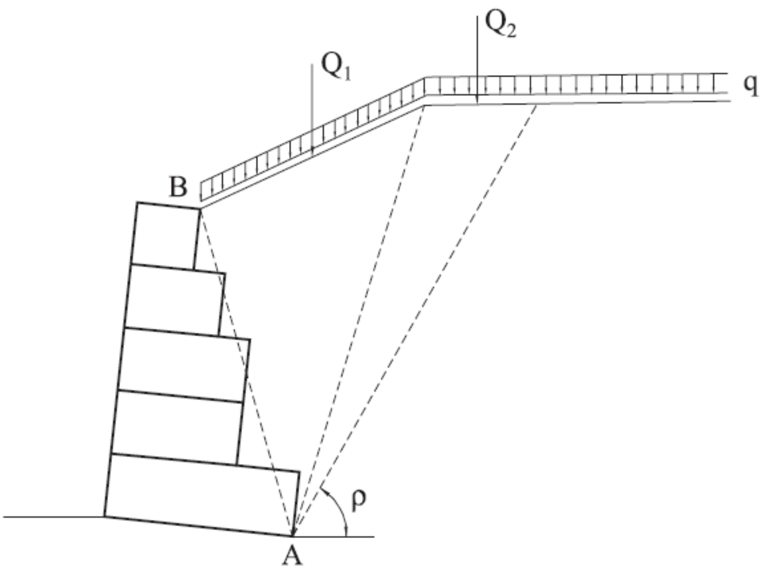


Figure 41 – Distributed load over backfill.

The active thrust "Ea" for each of the wedges is determined by:

$$E_a = (P + Q).\frac{\text{sen}(\rho - \phi)}{\text{sen}(\alpha + \rho - \phi - \delta)} \quad [105]$$

For the determination of the point of application of the obtained active thrust, the effects of the own weight of the soil are separated from the effect of the load:

$$E_{as} = P.\frac{\text{sen}(\rho - \phi)}{\text{sen}(\alpha + \rho - \phi - \delta)} \quad [106]$$

$$E_{aq} = Q.\frac{\text{sen}(\rho - \phi)}{\text{sen}(\alpha + \rho - \phi - \delta)} \quad [107]$$

The point of application of the effect of the own weight of the soil "Eas" is determined as in the previous item, while the effect of the distributed load "Eaq" is determined in an analogous way through a parallel to the rupture surface from the center of gravity of the point of application of the resulting "Q" load.

Load line on the embankment: Another situation that can occur is the application of a load line "Q" parallel to the load structure on the solid one, as shown in figure 42.

In this case, for wedges defined by rupture surfaces terminating at a point prior to the point of application of "Q" the load shall not be considered in the balance of forces:

$$E_{as} = P \cdot \frac{\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\alpha + \rho - \phi - \delta)} \qquad [\ 108\ ]$$

While adding the value of the load line "Q" to the value of the "P" weight of the wedges defined by bursting surfaces having their upper end at a point beyond the point of application of "Q":

$$E_a = (P + Q) \cdot \frac{\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\alpha + \rho - \phi - \delta)} \qquad [\ 109\ ]$$

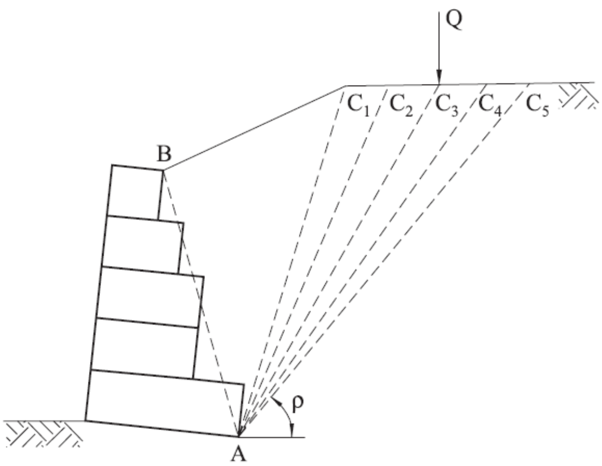


Figure 42 - Line Load over backfill

The variation curve of "Ea" with the position of the rupture surface will then show a discontinuity at the point of application of "Q", as shown in figure 43.

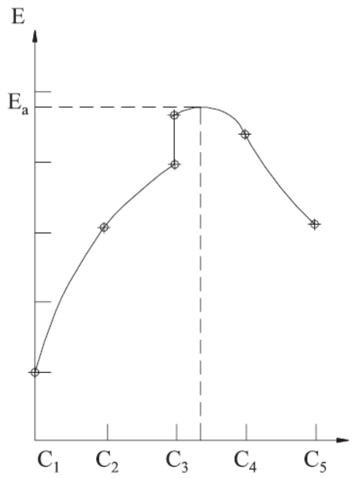


Figure 43 – Active Thrust variation due the inclination of failure surface.

If the maximum of the "Ea" variation curve occurs at a point prior to the point of discontinuity, the load line will have no influence on the active thrust, otherwise the effects of the "Eas" soil weight and the load line "EaQ" must be separated through the balance of the critical wedge:

$$E_{as} = P \cdot \frac{\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\alpha + \rho - \phi - \delta)} \qquad [\ 110\ ]$$

$$E_{aQ} = Q \cdot \frac{\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\alpha + \rho - \phi - \delta)} \qquad [\ 111\ ]$$

The point of application of "Ea" is determined by drawing a parallel to the rupture surface by the center of gravity "G" of the critical wedge.

For the determination of the point of application of "EaQ", from the point of application of "Q" one draws parallel to the rupture surface and a line with slope "φ" in relation to the horizontal. The intersection of these lines with the active thrust application surface defines the "N" and "M" points, respectively, as shown in Figure 44. The "EaQ" application point is located at a distance from the "M" point.

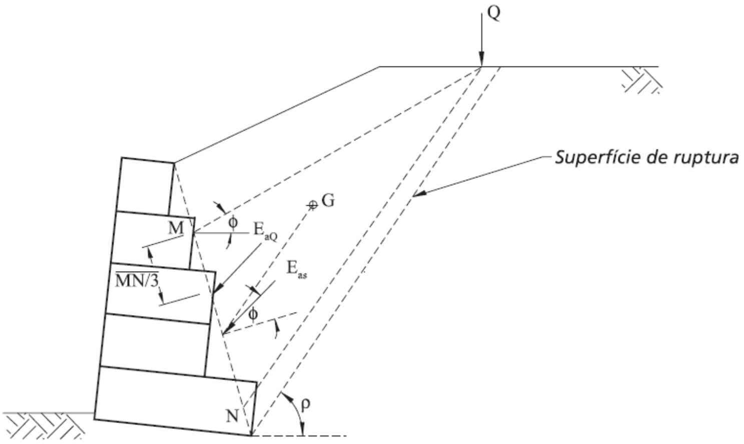


Figure 44 – Point of application of EAS and EAQ.

Cohesive backfill: When some cohesion is considered in the ground, it is necessary to consider the occurrence of traction cracks filled with water in the solid mass. The depth "z0" of these slits is given by:

$$z_0 = \frac{2.c}{\gamma} \cdot \frac{1}{\tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right)} \quad [\,112\,]$$

If there is a uniform overload "q" distributed over the mass, the depth "z0" must be reduced to:

$$z_0 = \frac{2.c}{\gamma} \cdot \frac{1}{\tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right)} - \frac{q}{\gamma} \quad [\,113\,]$$

The force applied by the water "Fw" against the walls of tension crack is:

$$F_w = \frac{1}{2} \cdot \gamma_a \cdot z_0^2 \quad [\,114\,]$$

And the force "C" due to soil cohesion is given by this cohesion "c" multiplied by the area of the rupture surface, as shown in figure 45.

The equilibrium of the forces that act on the wedge of soil allows the determination of "Ea" for each wedge analyzed:

$$E_a = \frac{P.\text{sen}\,(\rho - \phi) + F_w.\cos\,(\rho - \phi) - C.\cos\,\phi}{\text{sen}(\alpha + \rho - \phi - \delta)} \quad [\,115\,]$$

Once the maximum "Ea" value and the critical burst surface have been determined, the point of application of the soil effect on the thrust shall be situated at a "H / 3" height from the base of the wall. The point of application of the effects of eventual overloads is determined as in the previous items.

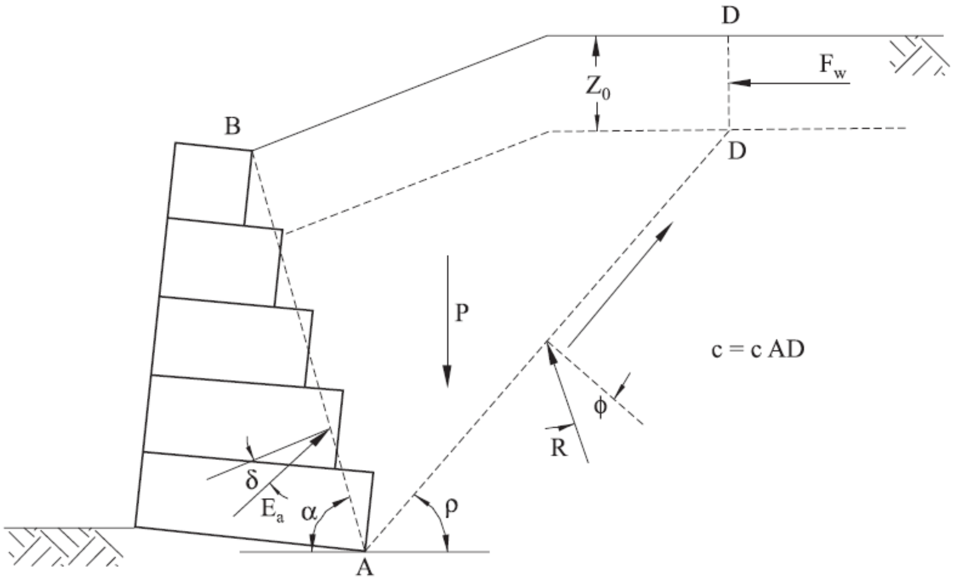


Figure 45 – Cohesive Soil

Mass partially submerged: If the solid mass is partially submerged, but there is no water percolation through it, it is enough to consider the specific gravity submerged "γ" of the soil below the water level for the calculation of the weight of each of the rupture wedges.

In this way each of the analyzed wedges is divided into two parts. One of them situated above the water level and the other situated below it. The weight of the first portion is determined using the natural specific weight "γ" of the soil, and the weight of the second portion is determined using the submerged specific gravity "γ". If the value of "γ" is unknown, it can be estimated by:

$$\gamma' = \gamma - (1 - n)\,.\gamma_w \qquad [\ 116\ ]$$

Where "n" is the porosity of the soil and "γw" is the specific weight of the water. One can adopt "n = 0.2 to 0.3".

In order to determine the point of application of the active thrust, it is necessary to determine the position of the center of gravity "G" of the critical wedge taking into account this difference in the value of the specific weights of the soil above and below the water level .

Mass with water percolation: When the solid mass is subject to water percolation it is necessary to take into account the effect of the percolation forces on the active thrust. For this it is necessary to trace the flow network through the mass, as shown in figure 3.7.13.

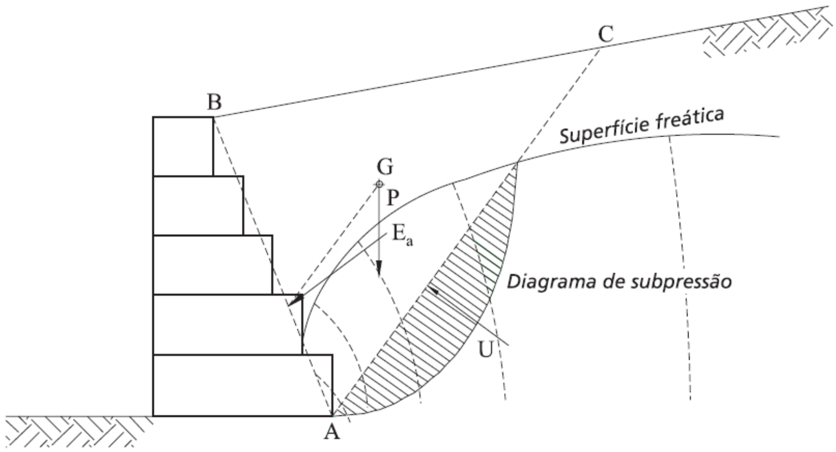


Figure 46 – Mass with water percolation

For each of the rupture surfaces analyzed, the diagram of subpressions acting on it is plotted and then the force "U" is determined by the water pressure along the rupture surface. The value of "U" is given by the area of the subpressure diagram multiplied by "γw".

When calculating the "P" weight of each wedge, the saturated specific weight "γsat" of the soil shall be used for the part of the wedge that is below the water table. If the value of "γsat" is not available, it can be estimated by:

$$\gamma_{sat} = \gamma + n \cdot \gamma_w \qquad [\ 117\ ]$$

Where "n" is the porosity of the soil. The value of "n" can be adopted as "n = 0.2".

There is a simplified alternative for the determination of "P" and "U". A specific mean weight is adopted for the soil and the value of "P" is calculated as if the soil were homogeneous. The value of the force "U" is then taken as proportional to the value of "P":

$$U = r_u \cdot P \qquad [\ 118\ ]$$

The value of "ru" depends on the height of the groundwater table in the mass and is normally between 0.2 and 0.5.

The thrust value "Ea" for each of the rupture surfaces analyzed is given by the equilibrium of the forces acting on the wedge and results in:

$$E_a = \frac{P.\ sen(\rho - \phi) + U.\ sen\ \phi}{sen(\alpha + \rho - \phi - \delta)} \qquad [\ 119\ ]$$

The point of application of the maximum active thrust "Ea" is determined by a parallel to the critical rupture surface passing through the center of gravity "G" of the ground wedge formed by it as in the previous items.

Seismic effect: The seismic effect is determined in the equilibrium limiting method, considering the balance of forces of each of the rupture wedges two additional forces: a horizontal force "H = Ch.P" and a vertical force "V = Cv.P ", Where" Ch "and" Cv "are horizontal and vertical acceleration coefficients respectively.

The values of "Ch" and "Cv" are given according to the seismic risk of the place where the wall is built and are specified by standards that vary according to the country. In most cases the value of the vertical coefficient "Cv" is considered null because it tends to decrease the seismic effect.

The balance of forces of each wedge determines the value of "Ea":

$$E_a = P.\frac{(1 - C_v).sen(\rho - \phi) + C_h.sen(\rho - \phi)}{sen(\alpha + \rho - \phi - \delta)} \qquad [\ 120\ ]$$

After determining the maximum "Ea" and the position of the critical rupture surface, given by "ρcrit", the static effect "Eas" can be separated from the total thrust:

$$E_{as} = P.\frac{(1 - C_v).\ sen\ (\rho_{crit} - \phi)}{sen(\alpha + \rho_{crit} - \phi - \delta)} \qquad [\ 121\ ]$$

The seismic effect "Ead" is then determined by:

$$E_{ad} = E_a - E_{as} \qquad [\ 122\ ]$$

The point of application of "Eas" is determined as in the previous items, while the point of application of "Ead" is situated "2H / 3" of the base of the wall.

6.2.4. Passive Thrust Calculation

The passive thrust "Ep", available in front of the retaining wall when the height of the ground "h" in front of the wall is higher than the base support, can be determined by Rankine's theory.

For non-cohesive soils this thrust is given by:

$$E_p = \frac{1}{2}.\gamma.h^2.K_p.\cos i \qquad [\ 123\ ]$$

Where:

$$K_p = \frac{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} \qquad [\ 124\ ]$$

And "i" is the slope of the soil surface in front of the wall, as shown in figure 47.

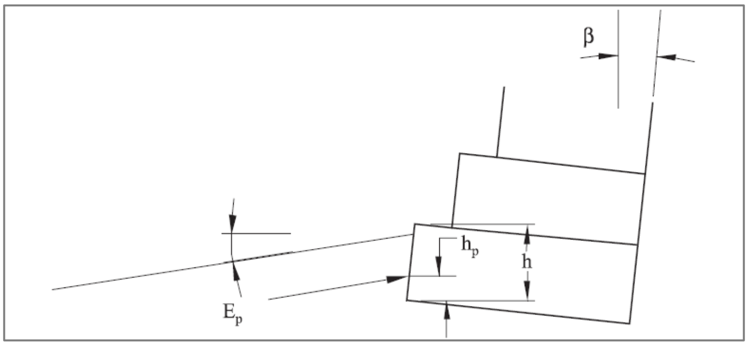


Figure 47 – Active Thrust Calculation

The application point of "Ep" is located at a "h / 3" height of the base of the wall and its direction is parallel to the surface of the ground in front of the wall.

If the surface of the ground in front of the wall is horizontal "i = 0", the value of "Ep" is:

$$E_p = \frac{1}{2}.\gamma.h^2.K_p \qquad [\ 125\ ]$$

$$K_p = \tan_2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) = \frac{1 + \text{sen}\ \phi}{1 - \text{sen}\ \phi} \qquad [\ 126\ ]$$

If the retaining wall is partially submerged and the soil in front of the wall is below the water level, the value of the submerged specific weight "γ'" is used in the calculation of "Ep".

5.2.5 Cohesive Soil

When the ground in front of the wall is cohesive and "i = 0", the passive thrust can be determined by:

$$E_p = \frac{1}{2}.\gamma.h^2.K_p + 2.c.h.\sqrt{K_p} \qquad [\ 127\ ]$$

And the value of "Kp" is calculated as in the previous item. The application point of "Ep" in this case is given by:

$$h_p = \frac{\gamma.h^3.K_p / 6 + c.h^2.\sqrt{K_p}}{E_p} \qquad [\ 128\ ]$$

In the case of "i> 0", first determine the value of the available passive pressure "pO" on the ground surface at the front of the wall and the available passive pressure "ph" at depth "h".

The pressure "pO" is given by:

$$p_o = \frac{2.c.\cos\phi.\cos i}{1 - \text{sen}\ \phi} \qquad [\ 129\ ]$$

And the pressure "ph" is given by:

$$p_h = \frac{o + \sqrt{o^2 - (1 + \tan^2 i).(o^2 - r^2)}}{(1 + \tan^2 i).\cos i} \qquad [\ 130\ ]$$

Where:

$$o = \frac{\sigma + c.\text{sen}\ \phi.\cos\phi + \sqrt{(\sigma + c.\text{sen}\ \phi.\cos\phi)^2 - \cos^2\phi.(\tau^2 + \sigma^2 - \cos^2\phi)}}{\cos^2\phi} \qquad [\ 131\ ]$$

$$r = o.\text{sen}\ \phi + c.\cos\phi \qquad [\ 132\ ]$$

$$\sigma = \gamma.h.\cos^2 i \qquad [\ 133\ ]$$

$$\tau = \gamma.h.\text{sen}\ i.\cos i \qquad [\ 134\ ]$$

The passive thrust "Ep" results:

$$E_p = \frac{p_o + p_h}{2}.h$$ [135]

And its point of application is situated on:

$$h_p = \frac{p_o.h^2/2 + (p_h + p_o).h^2/6}{E_p}$$ [136]

5.2.6 Wall Weight Calculation

It is necessary to determine the weight of the reinforcement structure for the stability analyzes.

The "P" weight of the gabion wall is obtained by multiplying the area "S" shown in Figure 48 by the specific weight "γg" of the gabion filler material. The value of "γg" is obtained from the specific weight of the material composing the "γp" stones and the "n" porosity of the gabions:

$$\gamma_g = \gamma_p.(1-n)$$ [137]

And the weight is then given by:

$$P = \gamma_g \,.\, S = \gamma_p.(1-n).S$$ [138]

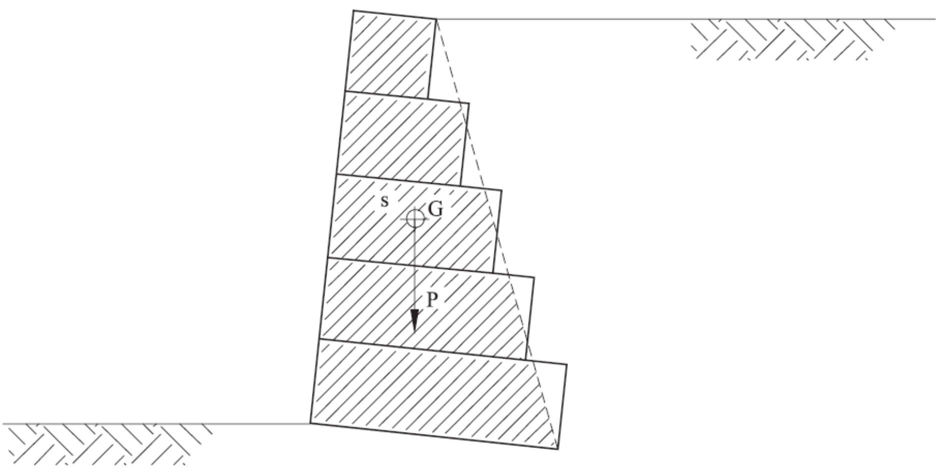


Figure 48 – Wall Weight Calculation

Values of "γp" for some types of rock can be found in table 2.

It is also necessary to determine the position of the center of gravity "G" of the wall, which, in this case, coincides with the center of gravity of area "S". For this determination, "S" is divided into triangles and the "Si" area and the center of gravity coordinates "Gi" of each of these triangles are determined.

The coordinates of the center of gravity of each triangle are the averages of the coordinates of each of its three vertices.

Table 2 – Rock Unit Weight

Type of Rock	Unit Weight [kN/m³]
Basalt	2.5-3.3
Diorito	2.5-3.3
Gabro	2.7 - 3.1
Gnaisse	2.5 - 3.0
Granite	2.6-3.3
Limestone	1.7 - 3.1
Marble	2.5-3.3
Quartzite	2.65

The coordinates of "G" are obtained from the weighted averages between areas and coordinates of the centers of gravity of each of the triangles.

If the wall is partially submerged, the submerged specific gravity of the gabions "$\gamma'g$" must be used for the part of the wall below the water level. The value of "$\gamma'g$" is given by:

$$\gamma'_g = \gamma_g - (1-n).\gamma_w = (1-n).(\gamma_p - \gamma_w) \quad [139]$$

When determining the center of gravity "G" of the wall, one must also take into account the specific weight difference between the part of the wall above and the part below the water level.

In cases where a seismic effect is to be considered, in addition to the weight "P" two inertia forces applied in "G" shall be applied to the wall: one horizontal "H" and the other vertical "V" given by:

$$H = C_h . P \quad [140]$$

$$V = C_v \cdot P \quad [141]$$

Where "Ch" and "Cv" are the horizontal and vertical acceleration coefficients associated with the site seismic risk.

7. Examples

7.1. Example 01

Wall Data:

Gabion filling material | Specific weight: γ_g = 25 kN/m³

Gabion porosity | n=30%

Reduction due the presence of the geotextile | 5%

Wall batter | β = 6°

Foundation allowable Bearing Capacity | 200 KPa

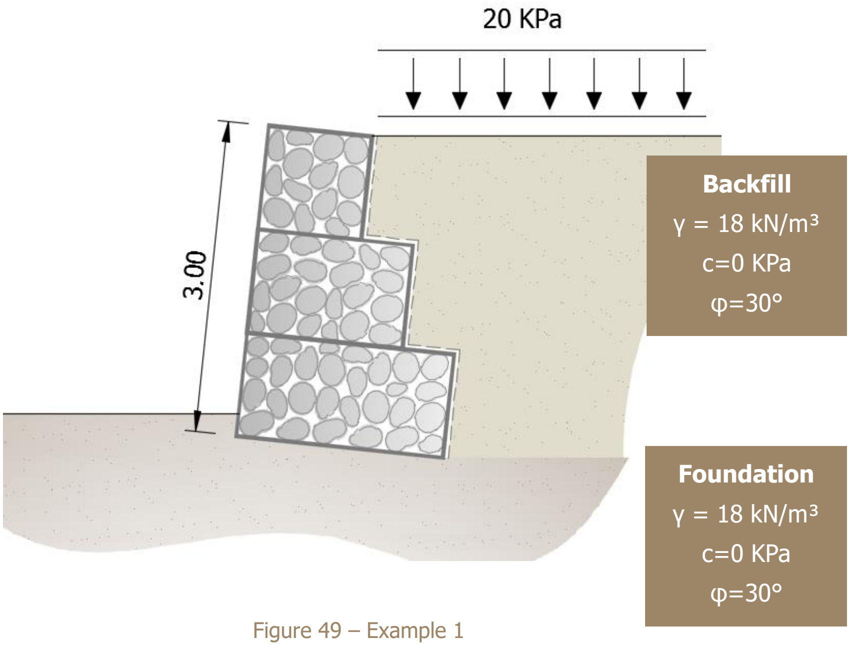


Figure 49 – Example 1

7.1.1. Example 01 – Active thrust calculation

a) Calculation of active thrust

The forces acting in the wedge are represented in the figure 50.

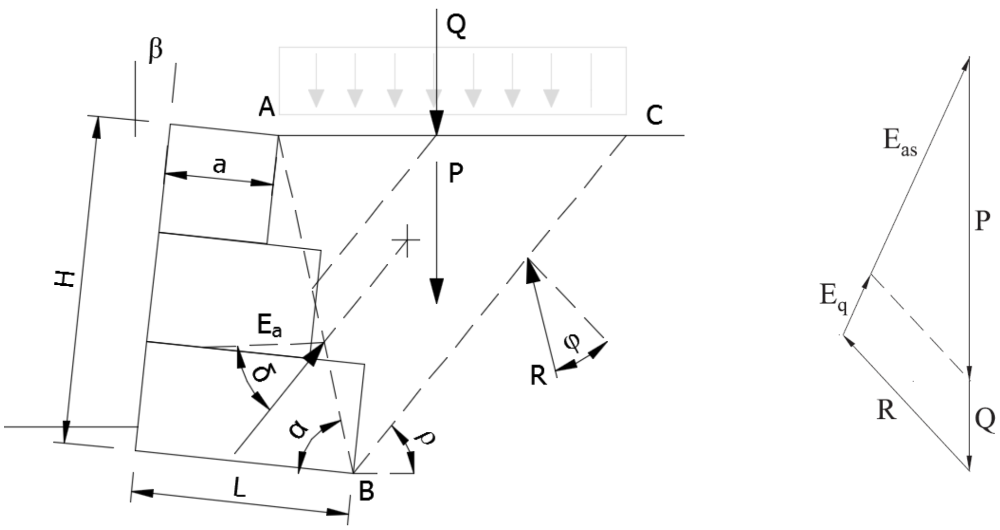


Figure 50 – Forces acting in the wedge

The angle δ, which is the interface angle between backfill and the wall can be assumed by 95% of φ due de presence of geotextile.

This example can be calculated by the limit equilibrium method, using Coulomb Theory. The maximum earth pressure can be calculated by the force equilibrium in the wedge ABC, by changing the length D (hence the angle ρ also will change).

The active thrust will give by:

$$E_a = \frac{P.\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\pi - \alpha - \rho + \phi + \delta)}$$

The angle "α" will be given by:

$$\alpha = \mathrm{atan}\left(\frac{H}{L-a}\right) + \beta$$

$$\alpha = \mathrm{atan}\left(\frac{3}{2-1}\right) + 6 = 77.57°$$

You may apply this equation varying the length "D":

At first, a value of **0.50** will be considered to D.

To simplify the calculation of wedge ABC, you may divide this wedge in two parts:

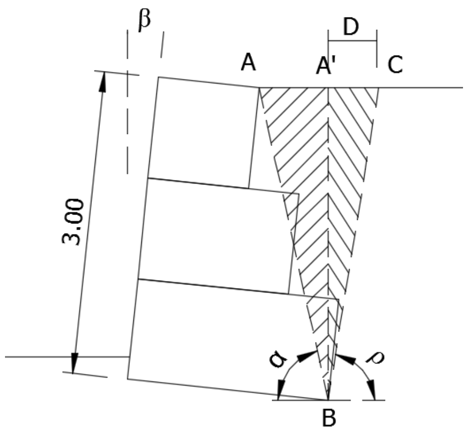


Figure 51 – Variation of distance "D"

The area of wedge A-A'-B will be given by:

$$AA'B = \frac{A'A \cdot A'B}{2}$$

Segment A'B:

$$AB = H \cdot \cos\beta - a \cdot \sin\beta + L \cdot \sin\beta$$

$$AB = 3 \cdot \cos(6) - 1 \cdot \sin(6) + 2 \cdot \sin(6)$$

$$AB = 3.09\text{m}$$

Segment A'A:

$$A'A = A'B \cdot \sin(90 - \alpha)$$

$$A'A = 3.09 \cdot \sin(90 - 77.57)$$

$$A'A = 0.68\text{m}$$

Calculation of the A-A'-B area:

$$AA'B = \frac{A'A \, . \, A'B}{2} = \frac{3.09 \, . \, 0.68}{2} = 1.05\text{m}$$

The area of wedge A'-B-C will be given by:

$$A'BC = \frac{A'B \, . \, A'C}{2} = \frac{3.09 \, . \, 0.50}{2} = 0.77\text{m}^2$$

Total wedge ABC area:

$$ABC = 1.05 + 0.77 = 1.82\text{m}^2$$

The total weight of the wedge will be:

$$P = ABC \, . \, \gamma = 1.82\text{m}^2 . \, 18 \, \frac{\text{kN}}{\text{m}^3} = 32.81 \text{ kN}$$

(Where "γ" is the unit weight of backfill)

The angle ρ can be calculated by:

$$\rho = \text{atan}\frac{A'B}{A'C} = \text{atan}\frac{3.09}{0.50} = 80.81°$$

b) Calculation of active thrust | Active Thrust due the soil weight | E_{AS}

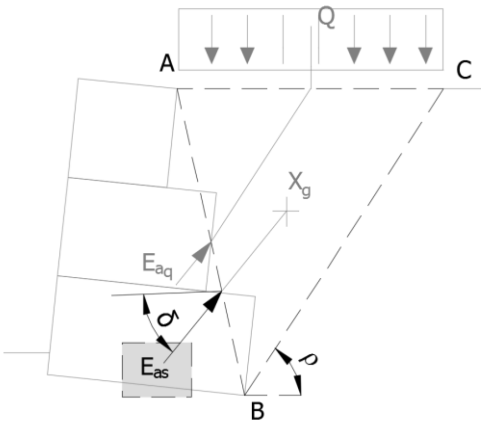


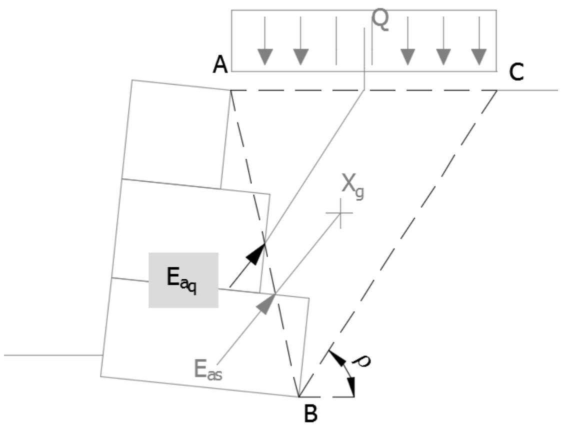
Figure 52 – Active thrust

The lateral earth pressure considering the distance D equal to 0.50m will be given by:

$$Eas = \frac{P.\sin(\rho - \varphi)}{\sin(\pi - \alpha - \rho + \varphi + \delta)}$$

$$Eas = \frac{32.81.\sin(80.81 - 30)}{\sin(180 - 77.57 - 80.81 + 30 + 28.5)} = 25.81\ kN/m$$

c) Calculation of active thrust | Active Thrust due the soil weight | E_{AQ}



The total weight caused by the resultant of load will be:

$$q.AC = 20\,.\,(0.68 + .5) = 23.62\ kN$$

The lateral earth pressure, caused by surcharge q, considering the distance D equal to 0.50m will be given by:

$$Eaq = \frac{23.62.\sin(80.81 - 30)}{\sin(180 - 77.57 - 80.81 + 30 + 28.5)} = 18.58\ kN/m$$

The total active thrust will be given by the sum of active thrust due the weight of soil and the active thrust due the load:

$$Ea = Eas + Eaq = 25.81 + 18.58 = \mathbf{44.39\ kN/m}$$

To continue the calculation, the same procedure must be applicate for other distances of D, until get the maximum value of earth pressure:

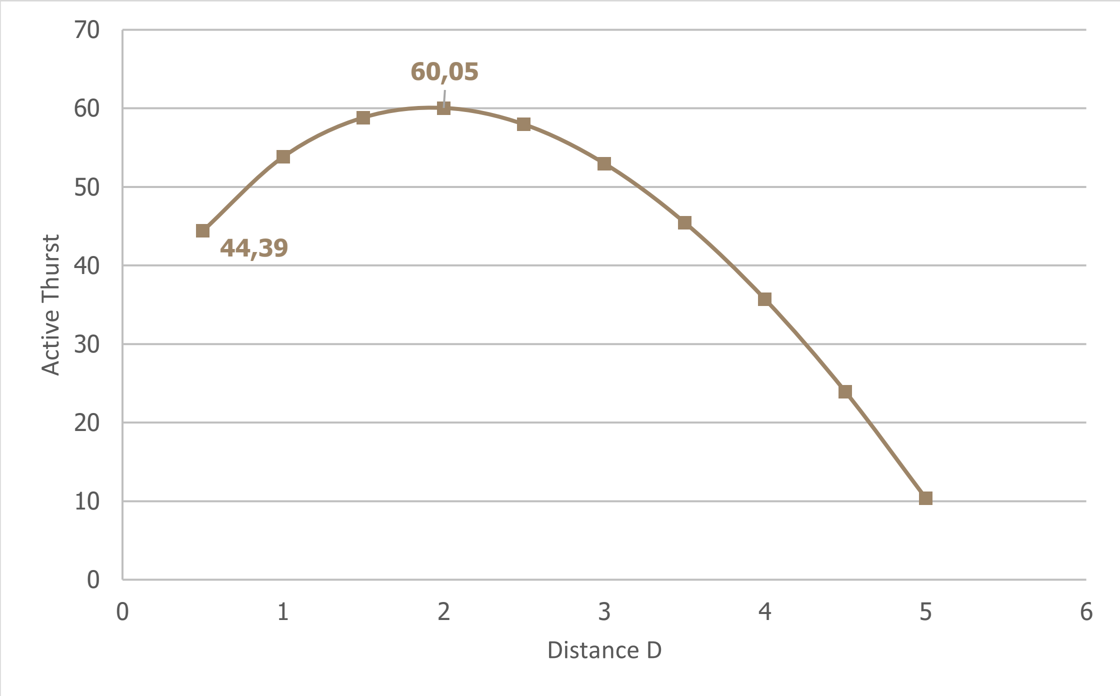


Table of results:

D [m]	ρ [°]	EAs [kN/m]	EAq [kN/m]	EA [kN/m]
0.5	80.80	25.81	18.58	44.39
1	72.06	31.30	22.53	53.82
1.5	64.09	34.21	24.63	58.84
2	**57.07**	**34.92**	**25.13**	**60.05**
2.5	51.01	33.71	24.26	57.96
3	45.83	30.81	22.17	52.99
3.5	41.42	26.44	19.03	45.47
4	37.67	20.76	14.94	35.70
4.5	34.46	13.92	10.02	23.94
5	31.70	6.04	4.35	10.39

Table 1 -Results of active thrust

The maximum earth pressure is **60.05 kN/m**. The angle **ρ** is **57.07°** and the distance D is 2m.

The application point of active thrust of soil can be obtained by the center of gravity of the critical wedge. After that, a parallel line to the angle ρ is project until the interface AB (Figure 53)

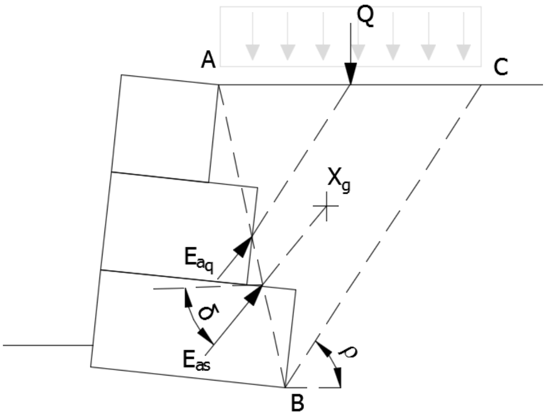


Figure 53 – Critical wedge ABC

First, the center of gravity of the wedge can be solve by the weighted average of the two triangles:

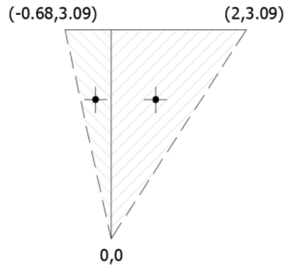


Figure 54 – Center of gravity of the wedge

The center of gravity of "x" will be given by:

$$Xg = \frac{\sum A.xg}{\sum A}$$

$$Xg = \frac{\left[\frac{(3.09\,.\,0.68)}{2}.\frac{-0.68}{3}\right] + \left[\frac{(3.09\,.\,2)}{2}.\frac{2}{3}\right]}{\frac{(3.09\,.\,0.68)}{2} + \frac{(3.09\,.\,2)}{2}}$$

$$Xg = 0.445\text{m}$$

The center of gravity of "y" will be given by:

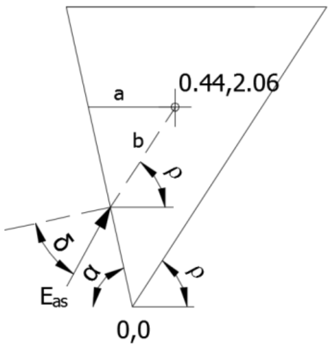
$$Yg = \frac{\sum A.yg}{\sum A}$$

$$Yg = \frac{\left[\frac{(3.09\,.\,0.68)}{2}.\frac{3.09}{3}.2\right] + \left[\frac{(3.09\,.\,2)}{2}.\frac{3.09}{3}.2\right]}{\frac{(3.09\,.\,0.68)}{2} + \frac{(3.09\,.\,2)}{2}}$$

$$Yg = 2.06m$$

d) Height of active Thrust | Soil

Then, the height of active thrust can be determined by projecting the center of gravity on the interface of the wall:



First, you need find the value of distance "a":

$$a = [H \,.\tan\alpha - (H - ycg).\tan(90 - \alpha)] + xcg$$

$$a = [3.09 \,.\tan 77.57 - (3.09 - 2.03).\tan(90 - 77.57)] + 0.445$$

$$a = 0.89m$$

Then, you may find the distance "b" by using the law of sins:

$$b = \frac{a.\sin\alpha}{\sin(180 - \rho - \alpha)}$$

$$b = \frac{0.89.\sin(77.57)}{\sin(180 - 57.07 - 77.57)}$$

$$b = 1.22m$$

After that, the height of active thrust will be:

$$YEa = Yg - \; b\,.\sin\rho$$

$$YEa = 2.06 - \; 1.22\,.\sin 57.07$$

$$YEa = 1.04m$$

The distance "x" will be:

$$XEa = Xg - \ b \,.\cos\rho$$

$$YEa = 0.44 - \ 1.22 \,.\cos 57.07$$

$$XEa = -0.22m$$

e) Height of active Thrust | Load

The height of the thrust caused by the load can be determined by projecting a parallel line of the angle ρ of the center of the load.

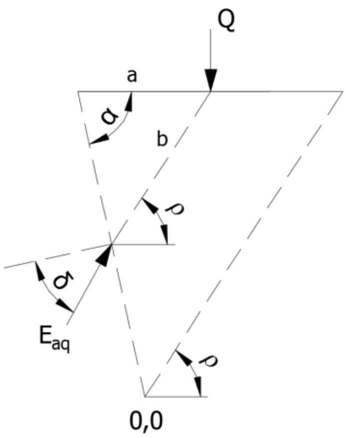


Figure 55 – Point of application – Load Active thrust

First, you may find the distance "b". The distance "a" corresponding to the half-distance of the top line of the wedge (2.68/2 = 1.34m).

Applying the law of sins:

$$b = \frac{a.\sin\alpha}{\sin(180 - \rho - \alpha)}$$

$$b = \frac{1.34.\sin(77.57)}{\sin(180 - 57.07 - 77.57)}$$

After that, the height (y coordination) of active thrust will be:

$$\mathrm{YEaq} = \mathrm{H} - \mathrm{b}\,.\sin\rho$$

$$\mathrm{YEaq} = 3.09 - 1.84\,.\sin(57.07)$$

$$\mathrm{XEaq} = [\frac{2.68}{2} - 0.68\,]\;\mathrm{b}\,.\cos\rho$$

$$\mathrm{XEaq} = -0.34\mathrm{m}$$

f) Coordinates of the height of the total active thrust

The total active thrust can be calculated by the weighted average of the two parcels (Eas and Eaq)

E_{AS}	34.92 kN/m	**YEas**	1.04m	**XEas**	-0.22m
E_{AQ}	25.13 kN/m	**YEaq**	1.54m	**XEaq**	-0.34m

Then, the height will be calculated by:

$$\mathrm{YEa} = \frac{34.92\,.\,1.04 + 25.13\,.\,1.54}{34.92 + 25.13}$$

$$\mathrm{YEa} = 1.249\mathrm{m}$$

Then, the coordinate x will be calculated by:

$$\mathrm{XEa} = \frac{34.92\,.\,(-0.22) + 25.13\,.\,(-0.34)}{34.92 + 25.13}$$

$$\mathrm{XEa} = -0.27\mathrm{m}$$

g) Adjusting coordinates for the analysis

Before starting the analysis, the reference point may be considering in the first box of the wall (Such as Gawac GSC considerers).

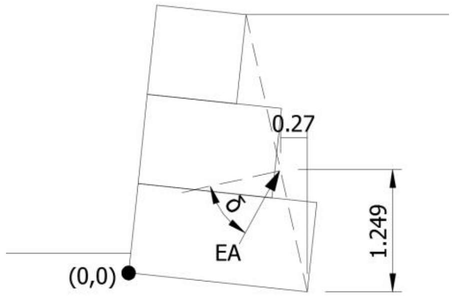


Figure 56 – Reference 0,0 coordinates

By considering the X,Y reference at the of the wall, the coordinates of active thrust will be (Figure 41):

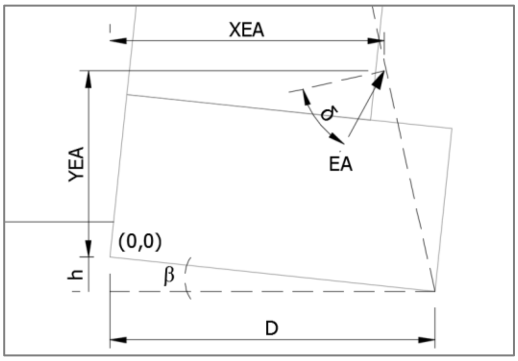


Figure 57 – Distances to get the reference point

The YEa, according to the 0,0 on the base of the wall will be:

$$\mathrm{YEa} = 1.249 - 2 \, . \sin\beta$$

$$\mathrm{YEa} = 1.249 - 2 \, . \sin(6)$$

$$\mathrm{YEa} = 1.04\mathrm{m}$$

$$\mathrm{XEa} = 2. \cos(\beta) - 0.27$$

$$\mathrm{XEa} = 1.71\mathrm{m}$$

The direction of the active thrust referred to x, is according to the angles δ, β and α:

$$\theta = (90 - \beta) - (\alpha - \beta)$$

$$\theta = (90 - 6) - (77.57 - 6)$$

$\theta = 40.93°$

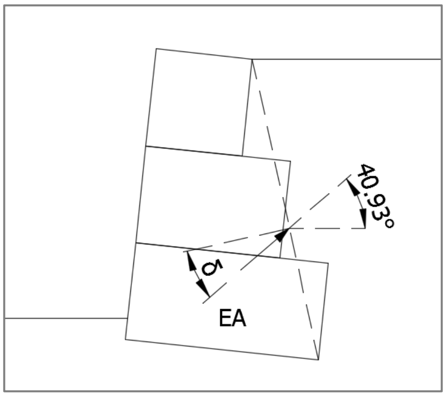
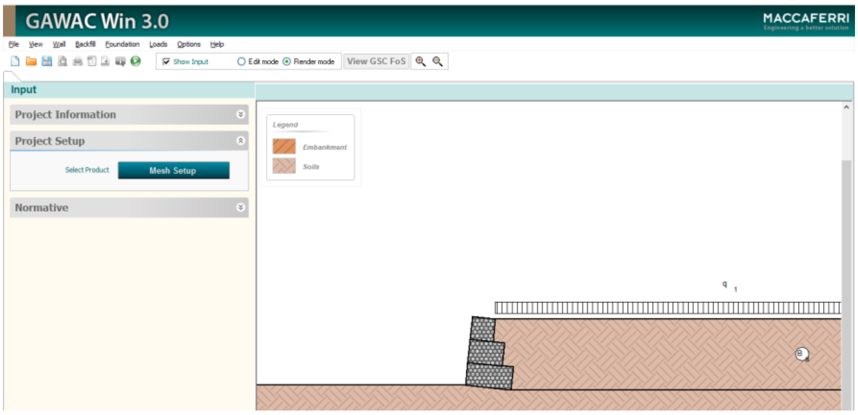


Figure 58 – Direction of active thrust

The values of active thrust can be calculated in the software Gawac (See Gawac Userguide).

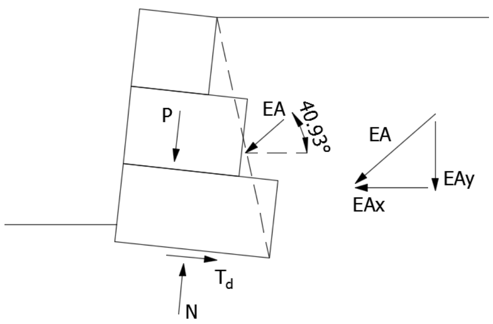


Active and Passive Thrust

Active Thrust [kN/m]	60.06
Point of application ref. to X axis [m]	1.71
Point of application ref. to Y axis [m]	1.04
Direction of the thrust ref. to X axis [m]	40.93
Passive Thrust [kN/m]	0.00
Point of application ref. to X axis [m]	0.00
Point of application ref. to Y axis [m]	0.00
Direction of the thrust ref. to X axis [m]	0.00

7.1.2. Example 01 – Sliding Check

a) Calculation of normal force on the base:



The Normal force will be given by:

$$N = P\,.\cos\beta + Ea.\,sen(\theta + \beta)$$

The force "P" is given by:

$$P = Ag\,.\,\gamma g\;(1 - n)$$

Where "γg" is the specific weight of the gabions (25kN/m³), "n" is the gabion porosity (n=30%) and Ag is the area of the cross section.

$$Ag = (1\,x1 + 1.5\;x\;1 + 2x1) = 4.5\;m^2$$

$$P = 4.5m^2.\,25kN/m^3\;(1 - 0.3) = 78.75\;kN$$

Then, N will be calculated by the equilibrium of forces:

$$N = 78.75\;kN\,.\cos(6) + Ea.\,sen(\theta + \beta)$$

$$N = 78.75\;kN\,.\cos(6) + 60.05\,.\,sen(46.93)$$

$$N = 122.16\;kN$$

b) Calculation of tangential force on base:

The force "Td" will be given by:

$$Td = N\,.\tan\delta^*$$

Where δ^* is the friction angle of the foundation soil.

$$Td = 122.16 \text{ x } \tan(30)$$

$$Td = 70.53 \text{ kN}$$

Then, the safety factor will be:

$$FS_{slid} = \frac{Td + P.\sin\beta}{Ea.\cos(\theta + \beta)}$$

$$FS_{slid} = \frac{70.53 + 78.75\,.\sin(6)}{60.05\,.\cos(40.93 + 6)}$$

$$\mathbf{FS_{slid} = 1.92}$$

c) Results in Gawac

Stability Analysis Results

Active and Passive Thrust

Active Thrust [kN/m]	60.06
Point of application ref. to X axis [m]	1.71
Point of application ref. to Y axis [m]	1.04
Direction of the thrust ref. to X axis [m]	40.93
Passive Thrust [kN/m]	0.00
Point of application ref. to X axis [m]	0.00
Point of application ref. to Y axis [m]	0.00
Direction of the thrust ref. to X axis [m]	0.00

Sliding

Normal force on the base [kN/m]	122.19
Point of application ref. to X axis [m]	0.76
Point of application ref. to Y axis [m]	-0.08
Horizontal active force [kN/m]	41.01
Horizontal resistance force [kN/m]	78.78
Sliding check	1.92

Overturning

Overturning Moment [kN/m x m]	46.98
Restoring Moment [kN/m x m]	139.36
Overturning check	2.97

Stresses Acting on Foundation

Eccentricity	0.24
Normal stress on outer border [kN/m²]	105.82
Normal stress on inner border [kN/m²]	16.37
Max. allowable stress on the foundation [kN/m²]	75.14
Base normal stress (left)	0.71
Base normal stress (right)	4.59

Overall Stability

Initial distance at pivot leftside [m]	
Initial distance at pivot rightside [m]	
Initial depth referred to base [m]	
Max depth referred to base [m]	
Center of the arch referred to X axis [m]	-0.09
Center of the arch referred to Y axis [m]	4.11
Overall Stability Check	1.36

7.1.3. Example 01 – Overturning Check

a) Calculation of active moment:

The active moment will be given by:

$$ME_{AH} = Ea\,.\sin(\theta).\,yEA$$

$$ME_{AH} = 60.05\,.\cos(40.93).\,1.04$$

$$ME_{AH} = 47.18\frac{kN}{m}.\,m$$

For this case, the resistant moments are related to the weight of the wall and the vertical parcell of active thrust.

b) Calculation of resistant moment:

Calculation of the center of gravity of gabion wall:

$$X'g = \frac{\sum Ai\,.\,xgi}{\sum A}$$

$$X'g = \frac{(2\text{ x }1) + (1.5\,.0.75) + (1\,.0.5)}{(2 + 1.5 + 1)}$$

$$X'g = 0.806m$$

$$Y'g = \frac{\sum Ai\,.\,ygi}{\sum A}$$

$$Y'g = \frac{(2\text{ x }.5) + (1.5\,.1.5) + (1\,.2.5)}{(2 + 1.5 + 1)}$$

$$Y'g = 1.17m$$

The coordinate “x” will be given by:

$$X'g = xg\,.\cos\beta + yg.\sin\beta$$

$$X'g = 0.806\,.\cos 6 + 1.17.\sin 6$$

$$X'g = 0.923m$$

The resistant moment of the wall will be:

$$\mathrm{Mp} = \mathrm{X'g}\,.\,\mathrm{P}.\cos\beta$$

$$\mathrm{Mp} = 0.923.\,78.75.\cos 6$$

$$\mathrm{Mp} = 72.295\,\frac{\mathrm{kN}}{\mathrm{m}}.\,\mathrm{m}$$

The resistant moment due the vertical parcel of active thrust will be:

$$\mathrm{M_{EAV}} = \mathrm{Ea}\,.\cos(\theta)\;.\,\mathrm{XEa}$$

$$\mathrm{M_{EAV}} = 60.05\,.\cos(40.93)\;.\,1.71$$

$$\mathrm{M_{EAV}} = 67.27\,\frac{\mathrm{kN}}{\mathrm{m}}.\,\mathrm{m}$$

The factor of safety against overturning will be:

$$\mathrm{FSovr} = \frac{\mathrm{Mp} + \mathrm{M_{EAV}}}{\mathrm{ME_{AH}}}$$

$$\mathrm{FSovr} = \frac{72.29 + 67.27}{47.18}$$

$$\mathbf{FSovr = 2.96}$$

c) Results in Gawac:

Stability Analysis Results

Active and Passive Thrust

Active Thrust [kN/m]	60.06
Point of application ref. to X axis [m]	1.71
Point of application ref. to Y axis [m]	1.04
Direction of the thrust ref. to X axis [m]	40.93
Passive Thrust [kN/m]	0.00
Point of application ref. to X axis [m]	0.00
Point of application ref. to Y axis [m]	0.00
Direction of the thrust ref. to X axis [m]	0.00

Sliding

Normal force on the base [kN/m]	122.19
Point of application ref. to X axis [m]	0.76
Point of application ref. to Y axis [m]	-0.08

Overturning

Overturning Moment [kN/m x m]	46.98
Restoring Moment [kN/m x m]	139.36
Overturning check	2.97

Stresses Acting on Foundation

Eccentricity	0.24
Normal stress on outer border [kN/m²]	105.82
Normal stress on inner border [kN/m²]	16.37
Max. allowable stress on the foundation [kN/m²]	75.14
Base normal stress (left)	0.71
Base normal stress (right)	4.59

7.1.4. Example 01 – Foundation Check

a) Eccentricity calculation

The distance "d", between the base point and the point of application of force "N".

$$d = \frac{Mp + MEa - MEah}{N}$$

$$d = \frac{72.29 + 67.27 - 47.18}{122.16}$$

$$d = 0.76m$$

Eccentricity calculation:

$$e = \frac{B}{2} - d = \frac{2}{2} - 0.71 = 0.24m$$

Eccentricity condition:

$$0.24 < \frac{2}{6}$$

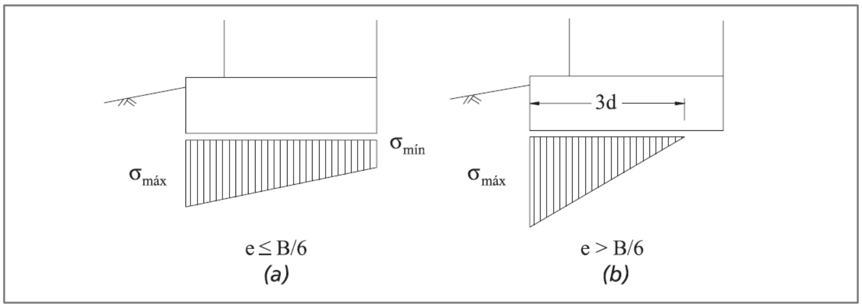


Figure 59 – Pression distribution on base

Then:

$$qmáx = \frac{N}{B}.(1 + 6.\frac{e}{B})$$

$$qmáx = \frac{122.16}{2}.(1 + 6.\frac{0.24}{2})$$

$$qmáx = 105.06\frac{kN}{m^2} < qadm = 200\ kN/m^2\ .:\ \textbf{Ok}$$

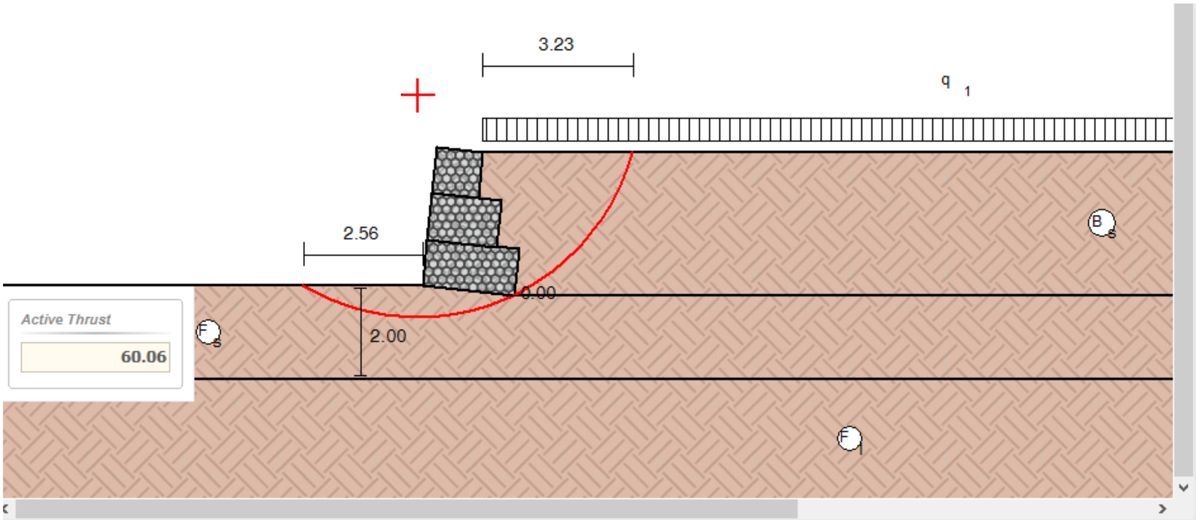
$$qmín = \frac{N}{B}.(1 - 6.\frac{e}{B})$$

$$qmín = \frac{122.16}{2}.(1 - 6.\frac{0.24}{2})$$

$$qmín = 17.10\frac{kN}{m^2} < qadm = 200\ kN/m^2\ .:\ \textbf{Ok}$$

7.1.5. Example 01 – Global Stability

The global stability can be verified using computers due the iteration equations and curves. The software Gawac provides this analysis.



Overall Stability

Initial distance at pivot leftside [m]	
Initial distance at pivot rightside [m]	
Initial depth referred to base [m]	
Max depth referred to base [m]	
Center of the arch referred to X axis [m]	-0.09
Center of the arch referred to Y axis [m]	4.11
Overall Stability Check	1.36

Figure 60 – Global stability results | Example 01

7.2. Example 02

Wall Data:

Gabion filling material | Specific weight: γ_g = 25 kN/m³

Gabion porosity | n=35%

Reduction due the presence of the geotextile | 5%

Wall batter | β = 0°

Foundation allowable Bearing Capacity | 170 KPa

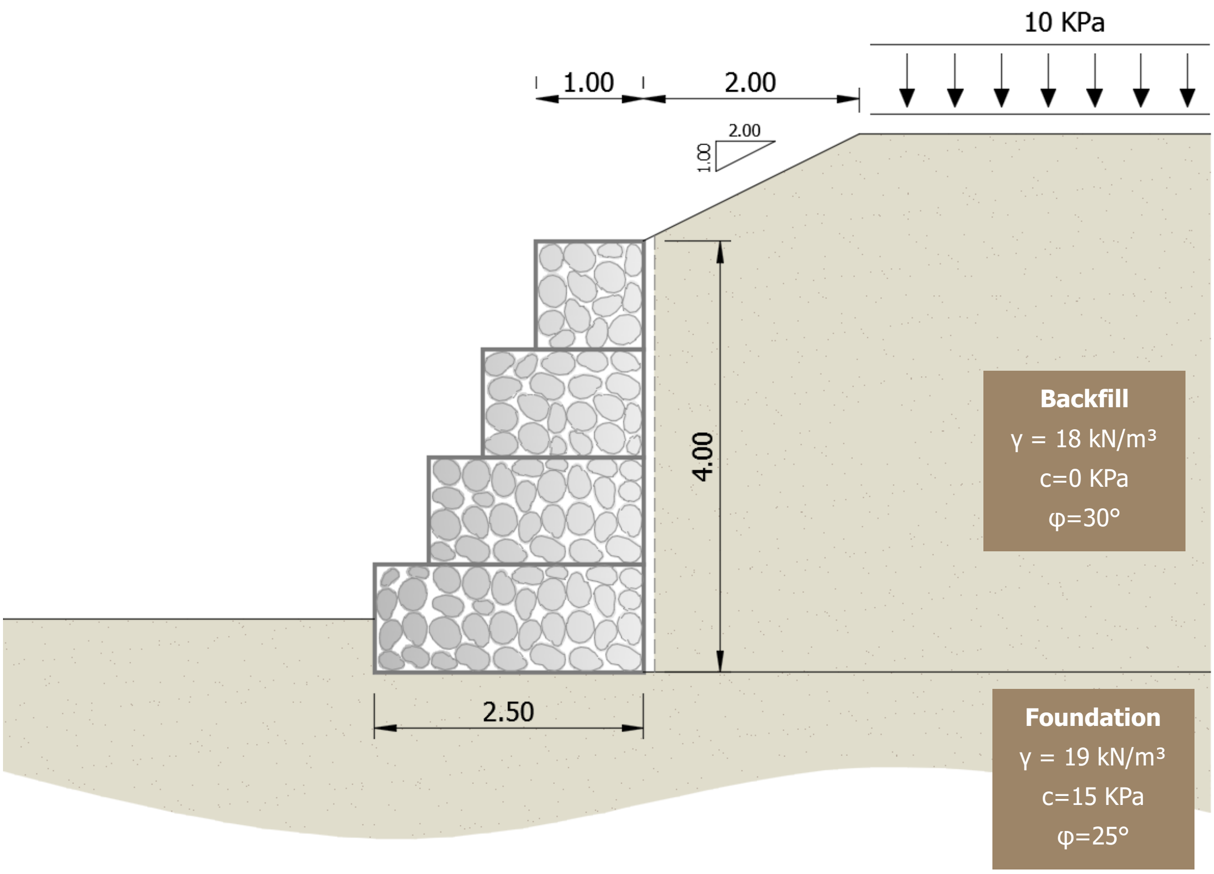


Figure 61 – Example 02

7.2.1. Example 02 – Active thrust calculation

a) Calculation of active thrust

The calculation of active thrust can be calculated by equilibrium limit method.

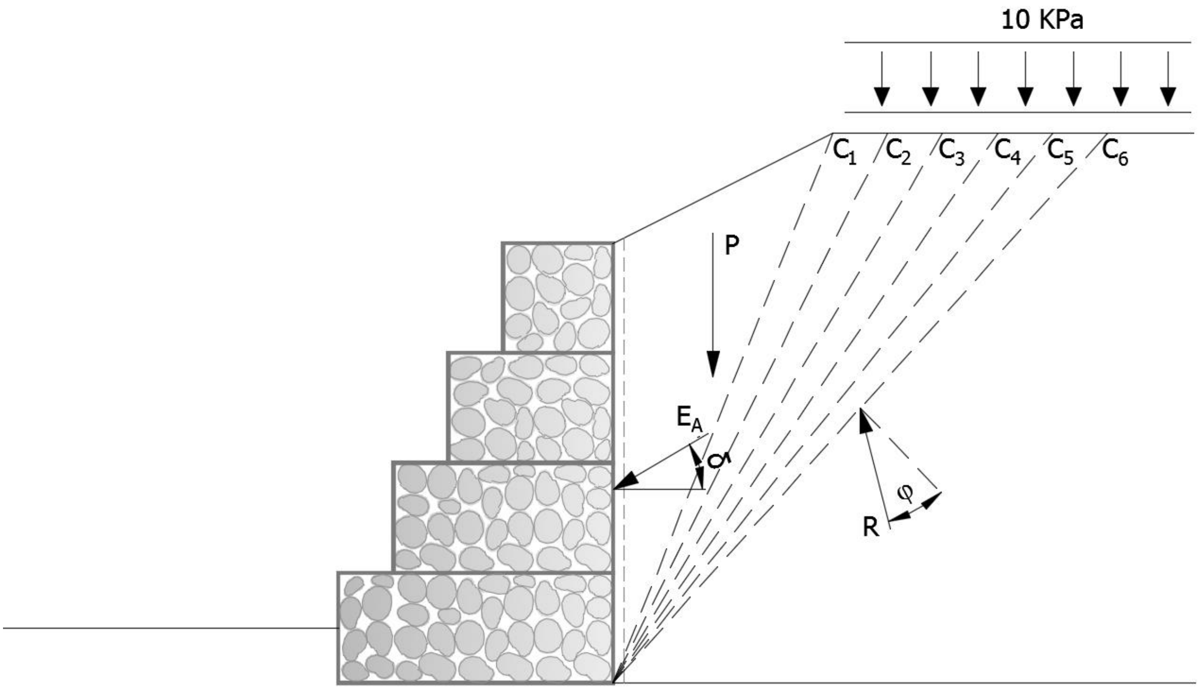


Figure 62 – Wedges of Limit equilibrium method

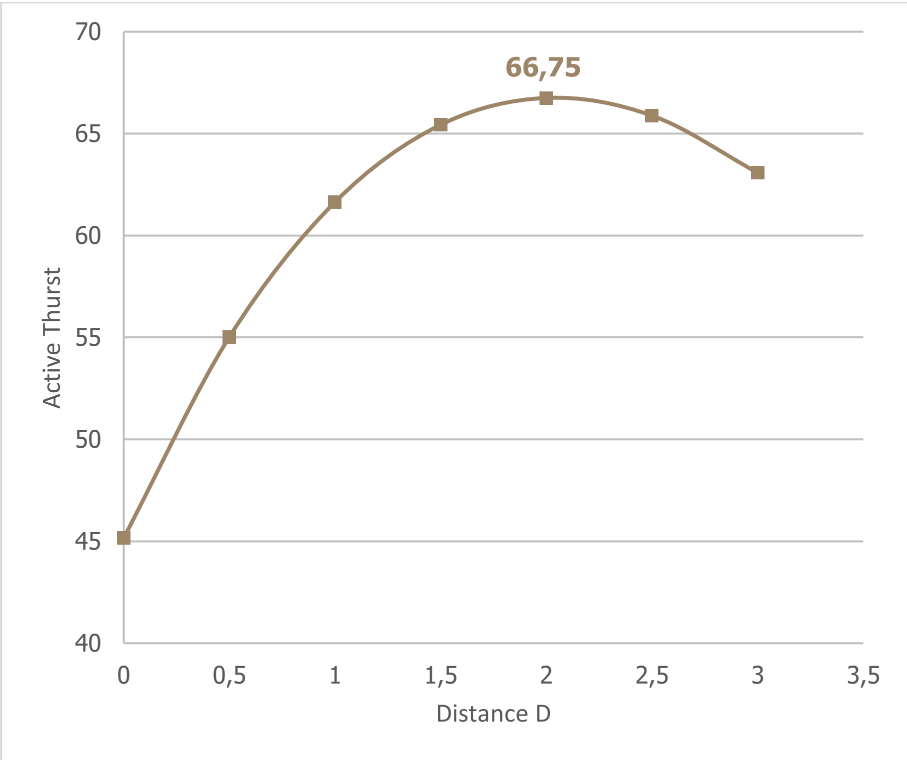
The active thrust will give by:

$$E_a = \frac{P.\mathrm{sen}(\rho - \phi)}{\mathrm{sen}(\pi - \alpha - \rho + \phi + \delta)} \quad [20]$$

The angle "α" is 90°.

The results are presented in the table 02:

Wedge	ρ [°]	D [m]	EAs [kN/m]	EAq [kN/m]	EA [kN/m]
1	68.20	0.00	45.17	0.00	45.17
2	63.43	0.50	52.26	2.77	55.03
3	59.04	1.00	56.79	4.85	61.64
4	55.01	1.50	59.08	6.35	65.44
5	**51.34**	**2.00**	**59.42**	**7.34**	**66.75**
6	48.01	2.50	58.02	7.86	65.88
7	45.00	3.00	55.10	7.99	63.08



The maximum earth pressure is **66.75 kN/m**. The angle **ρ** is **51.34°** and the distance D is 2m.

The application point of active thrust of soil can be obtained by the center of gravity of the critical wedge. After that, a parallel line to the angle ρ is project until the interface AB (Figure 63)

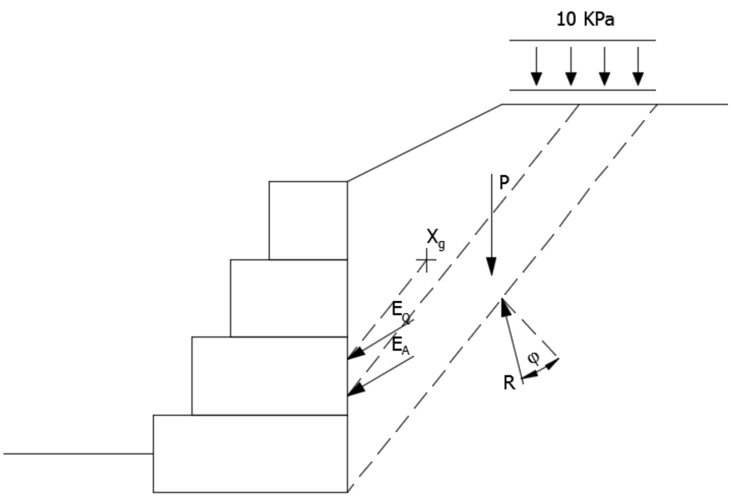


Figure 63 – Critical wedge

a) Center of gravity | Wedge

The center of gravity can be determined by dividing the wedge in two parts (I and II):

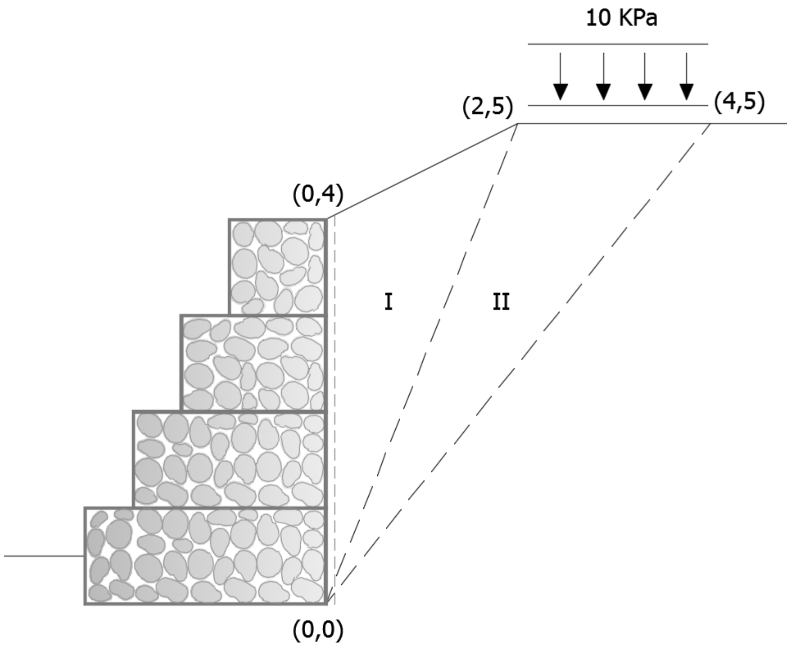


Figure 64 – Example 2 – Calculation of center of gravity

Then, the center of gravity and the area of which part will be given by:

Area 1:

$$Xg1 = \frac{0+0+2}{3}$$

$$Xg1 = 0.67m$$

$$Yg1 = \frac{0+4+5}{3}$$

$$Yg1 = 3m$$

$$A1 = 4m^2$$

Area 2:

$$Xg2 = \frac{0+2+4}{3}$$

$$Xg2 = 2m$$

$$Yg2 = \frac{0+5+5}{3}$$

$$Yg2 = 3.33m$$

$$A2 = 5m^2$$

Then, the center of gravity of the wedge will be:

$$Xg = \frac{Xg1.A1 + Xg2.A2}{A1 + A2}$$

$$Xg = \frac{0.67 \text{ x } 4 + 2 \text{ x } 5}{4+5}$$

$$Xg = 1.407m$$

Then, the center of gravity of the wedge will be:

$$Yg = \frac{Yg1.A1 + Yg2.A2}{A1 + A2}$$

$$Yg = \frac{3 \text{ x } 4 + 3.33 \text{ x } 5}{4+5}$$

$$Yg = 3.185m$$

b) Height of active thrust

Then, the height of active thrust can be determined by projecting the center of gravity on the interface of the wall:

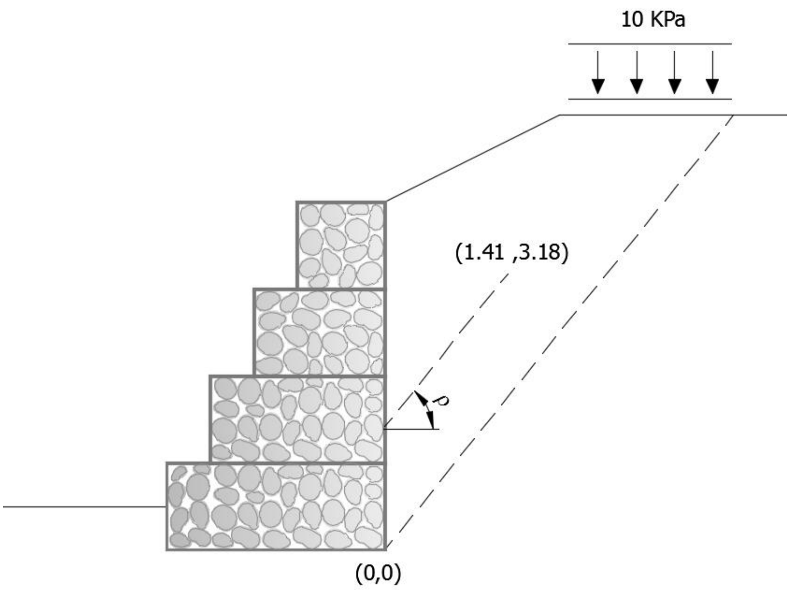


Figure 65 - Point of active thrust - Soil

$$YEA = Yg - Xg.\tan\rho$$

$$YEA = 1.43m$$

c) Height of active Thrust | Load

The height of the thrust caused by the load can be determined by projecting a parallel line of the angle ρ of the center of the load.

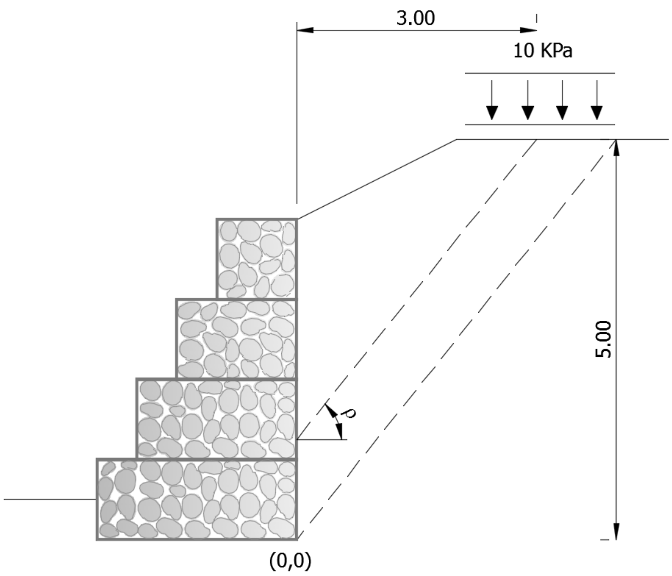


Figure 66 – Point of application – Load Active thrust

$$\text{YEAq} = 5 - 3 \,.\, \tan(51.34)$$

$$\text{YEAq} = 1.25\text{m}$$

d) Coordinates of the height of the total active thrust

The total active thrust can be calculated by the weighted average of the two parcels (Eas and Eaq)

E_{AS}	59.42 kN/m	**YEas**	1.43m
E_{AQ}	7.34 kN/m	**YEaq**	1.25m

Then, the height will be calculated by:

$$\text{YEa} = \frac{59.42 \text{ x } 1.43 + 7.34 \text{ x } 1.25}{59.42 + 7.34}$$

$$\text{YEa} = 1.42\text{m}$$

Then, the coordinate x will be calculated by:

$$\text{XEa} = 2.5\text{m}$$

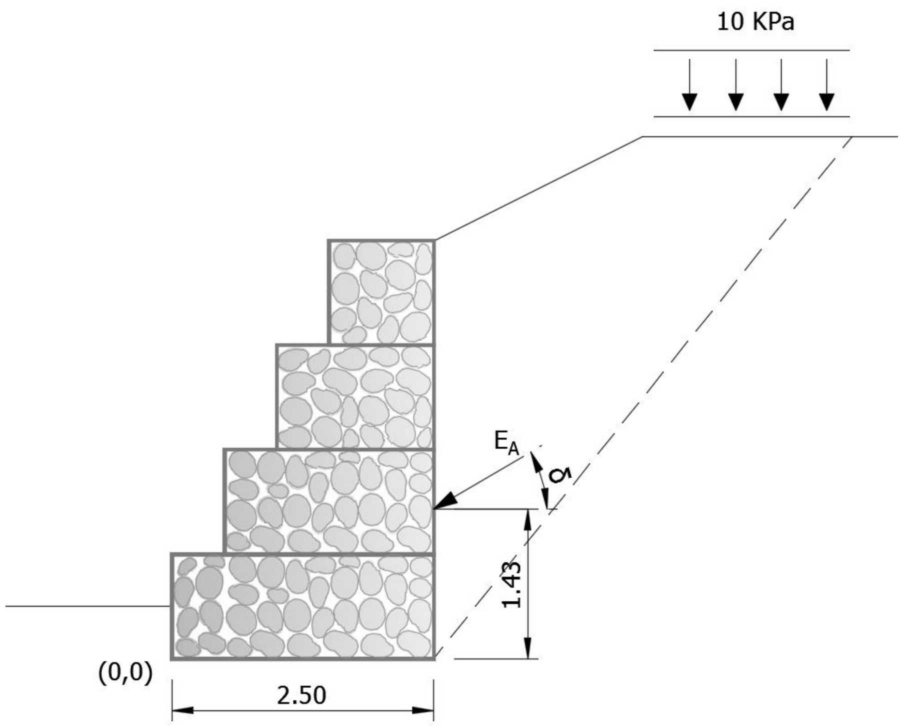


Figure 67 – Point of application | Active Thrust

7.2.2. Example 02 – Sliding Check

c) Calculation of normal force on the base:

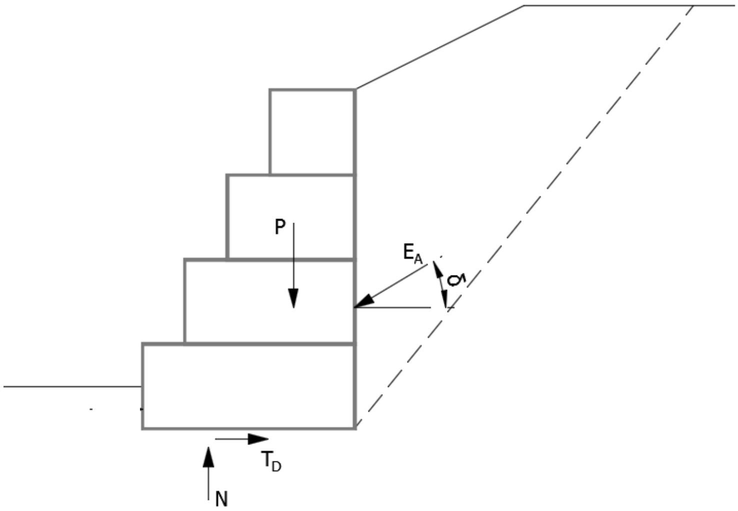


Figure 68 – Forces on base

The Normal force will be given by:

$$N = P + Ea.\,sen(\delta)$$

The force "P" is given by:

$$P = Ag\,.\,\gamma g\;(1 - n)$$

Where "γg" is the specific weight of the gabions (25kN/m³), "n" is the gabion porosity (n=35%) and Ag is the area of the cross section.

$$Ag = (1\,x1 + 1.5\;x\;1 + 2x1 + 2.5x1) = 7.0\;m^2$$

$$P = 7.0\;m^2.\,25kN/m^3\;(1 - 0.35) = 113.75\;kN$$

Then, N will be:

$$N = 113.75\;kN + 66.75.\,sen(28.5)$$

$$N = 145.60\;kN$$

d) Calculation of tangential force on base:

The force "Td" will be given by:

$$Td = N \cdot \tan \delta^*$$

$$Td = N \cdot \tan \delta^* + c \cdot B$$

Where "c" is the cohesion of foundation soil. By reason of security, this value can be divided by 2.

$$Td = 145.6 \cdot \tan(25) + \frac{15}{2} \cdot 2.5$$

$$Td = 86.64 \text{ kN/m}$$

Then, the safety factor will be:

$$FS_{slid} = \frac{Td}{Ea \cdot \cos(\delta)}$$

$$FS_{slid} = \frac{86.64}{66.75 \cdot \cos(28.5)}$$

$$\mathbf{FS_{slid} = 1.48}$$

c) Results in Gawac

Sliding

Normal force on the base [kN/m]	145.60
Point of application ref. to X axis [m]	1.15
Point of application ref. to Y axis [m]	0.00
Horizontal active force [kN/m]	58.65
Horizontal resistance force [kN/m]	86.64
Sliding check	1.48

Figure 69 – Sliding Check - Gawac

7.2.3. Example 02 – Overturning Check

d) Calculation of active moment:

The active moment will be given by:

$$ME_{AH} = Ea\,.\cos(\delta).\,yEA$$

$$ME_{AH} = 66.75\,.\cos(28.5).\,1.42$$

$$ME_{AH} = 83.30\frac{kN}{m}.m$$

e) Calculation of resistant moment:

Calculation of the center of gravity of gabion wall:

$$X'g = \frac{\sum Ai\,.\,xgi}{\sum A}$$

$$X'g = \frac{(2.5x\;1.25) + (2\;x\;1.5) + (1.5\,.\,1.75) + (1\,.\,2)}{(2.5 + 2 + 1.5 + 1)}$$

$$X'g = 1.53m$$

The resistant moment of the wall will be:

$$Mp = X'g\,.\,P$$

$$Mp = 1.53\;x\;113.75$$

$$Mp = 174.69\frac{kN}{m}.m$$

The resistant moment due the vertical parcel of active thrust will be:

$$M_{EAV} = Ea\,.\,sen(\delta)\;.\,XEa$$

$$M_{EAV} = 66.75\,.\,sin(28.5)\;.\,2.5$$

$$M_{EAV} = 79.62.\frac{kN}{m}.m$$

The factor of safety against overturning will be:

$$FSovr = \frac{Mp + M_{EAV}}{ME_{AH}}$$

$$FSovr = \frac{174.69 + 79.62}{83.30}$$

$$\mathbf{FSovr = 3.05}$$

7.2.4. Example 02 – Foundation Check

b) Eccentricity calculation

The distance “d”, between the base point and the point of application of force “N”.

$$d = \frac{Mp + MEa - MEah}{N}$$

$$d = \frac{174.69 + 79.62 - 83.3}{145.6}$$

$$d = 1.15m$$

Eccentricity calculation:

$$e = \frac{B}{2} - d = \frac{2.5}{2} - 1.15 = 0.10m$$

Eccentricity condition:

$$0.1 < \frac{2.5}{6}$$

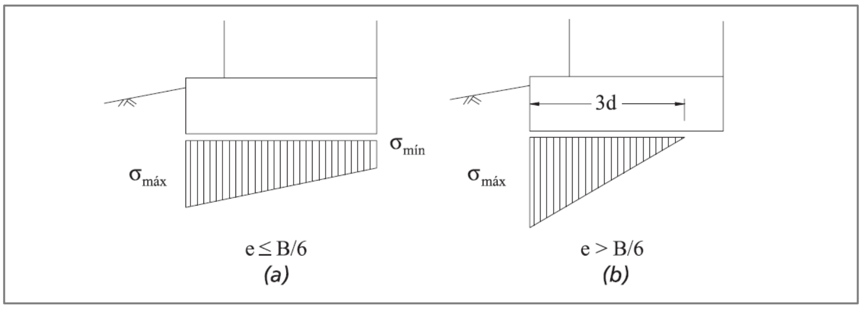


Figure 70 – Pression distribution on base

Then:

$$qmáx = \frac{N}{B}.(1 + 6.\frac{e}{B})$$

$$qmáx = \frac{145.6}{2.5}.(1 + 6.\frac{0.10}{2})$$

$$qmáx = 75.71\frac{kN}{m^2} < qadm = 170\ kN/m^2\ .:\ \textbf{Ok}$$

$$qmín = \frac{N}{B}.(1 - 6.\frac{e}{B})$$

$$qmín = \frac{145.6}{2.5}.(1 - 6.\frac{0.10}{2})$$

$$qmín = 40.77\frac{kN}{m^2} < qadm = 170\ kN/m^2\ .:\ \textbf{Ok}$$

7.2.5. Example 02 – Global Stability

The global stability can be verified using computers due the iteration equations and curves. The software Gawac provides this analysis.

Overall Stability

Initial distance at pivot leftside [m]	
Initial distance at pivot rightside [m]	
Initial depth referred to base [m]	
Max depth referred to base [m]	
Center of the arch referred to X axis [m]	0.96
Center of the arch referred to Y axis [m]	6.92
Overall Stability Check	1.77

Figure 71 – Global Stability – Example 02

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APPENDIX I – DESIGN WITH NORMATIVE

TECHNICAL NOTE

Ref. INT / TN – Rev. 0, Oct. 2019

NORMATIVE APPLICATION

GAWAC 3.0

SUMMARY

1. INTRODUCTION

The objective of this document is to explain how the partial factors of Standards can be applied in Gabion wall analysis (only in ULS - Ultimate Limit State).

Table 1 presents all partial factors:

Table 1 – Partial Factors description

Partial Factor	Related to	Partial Factor	Related to
$\gamma\varphi$	Friction angle	$\gamma\varphi$ ov	Friction angle | Overturning Analysis
γc	Cohesion	γc ov	Cohesion | Overturning Analysis
$\gamma c'$	Undrained Shear Strength	$\gamma c'$ ov	Undrained Shear Strength | Overturning Analysis
$\gamma G_{,unfav}$	Permanent Action (G) unfavorable	$\gamma G_{,unfav\ ov}$	Permanent Action (G) unfavorable | Overturning Analysis
$\gamma G_{,fav}$	Permanent Action (G) favorable	$\gamma G_{,fav\ ov}$	Permanent Action (G) favorable | Overturning Analysis
$\gamma Q_{,unfav}$	Variable Action (G) unfavorable	$\gamma Q_{,unfav\ ov}$	Variable Action (G) unfavorable | Overturning Analysis
$\gamma Q_{,fav}$	Variable Action (G) favorable	$\gamma Q_{,fav\ ov}$	Variable Action (G) favorable | Overturning Analysis
γRv	Bearing Resistance	γRv ov	Bearing Resistance | Overturning Analysis
γRh	Sliding Resistance	γRh ov	Sliding Resistance | Overturning Analysis
γRm	Overturning Resistance	γRm ov	Overturning Resistance | Overturning Analysis
$\gamma Re_{,intShear}$	Earth internal shear resistance	$\gamma Re_{,intShear\ ov}$	Earth internal shear resistance | Overturning Analysis
$\gamma Re_{,intComp}$	Earth internal compression resistance	$\gamma Re_{,intComp\ ov}$	Earth internal compression resistance | Overturning Analysis
$\gamma Re_{,Overall}$	Earth overall resistance	$\gamma Re_{,Overall\ ov}$	Earth overall resistance | Overturning Analysis
$\gamma G_{,wall}$	Wall weight	$\gamma G_{,wall\ ov}$	Wall weight | Overturning Analysis
$\gamma Q_{,water}$	Water thrust	$\gamma Q_{,water\ ov}$	Water thrust | Overturning Analysis

2. PARTIAL FACTORS

2.1 SOIL PROPERTIES

The soil properties are affected by the partial factors γ_φ and γ_C.

After inserting the soil properties in Gawawin (Figure 2), the software will reduce the friction angle by the Equation 1 and the soil cohesion by Equation 2:

$$\varphi_d = \operatorname{atan}\left(\frac{\tan(\varphi)}{\gamma\varphi}\right) \quad [1]$$

$$C_d = \frac{c}{\gamma_C} \quad [2]$$

Where:

φ	Characteristic soil friction angle
φ_d	Design soil friction angle
C	Characteristic soil cohesion
C_d	Design soil cohesion

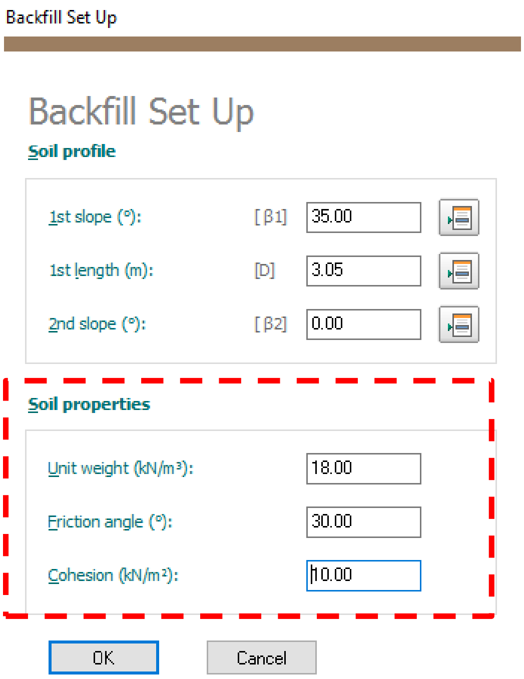


Figure 1 – Soil properties on GawacWin

2.2 SOIL ACTIONS

Soil actions are defined by the loads due the active and passive thrust. These kinds of loads are permanent. Permanent actions are affected by the partial factor γ_G.

2.2.1 Active Thrust

After calculating the active thrust due the soil, Gawac Increases this force by multiplying it by the partial factor γ_G – unfav, which means that this kind of load is unfavorable for the overall stability of the wall. Equation 3 (for granular soils) and Equation 4 represents how this partial factor is applied:

$$E_A = \frac{P.\sin(\rho - \varphi_d)}{\sin(\pi - \alpha - \rho + \varphi_d + \delta_d)} \quad [3]$$

$$E_{A,GD} = E_A \cdot \gamma_{G,UNFAV} \quad [4]$$

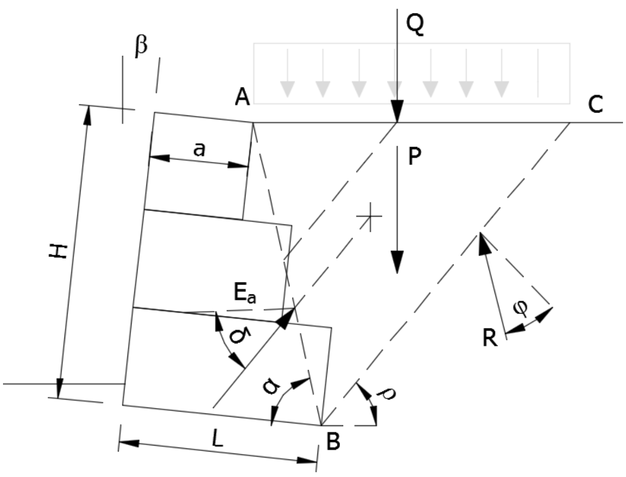


Figure 2 – Forces acting in the wedge

Where:

E_A	Active Thrust
P	Wedge Weight
φ_d	Design soil friction angle
ρ	Rupture line inclination
α	Angle between wall face and horizontal
δ_d	Angle between wall and backfill

2.2.2 Water Thrust

Whenever the wall has a phreatic surface, $\gamma_{Q,water}$ shall be applied in water unit weight.

$$\gamma_w = \gamma_w \cdot \gamma_{Q,WATER} \quad [5]$$

This value will affect the pore pressure force U (Figure 3):

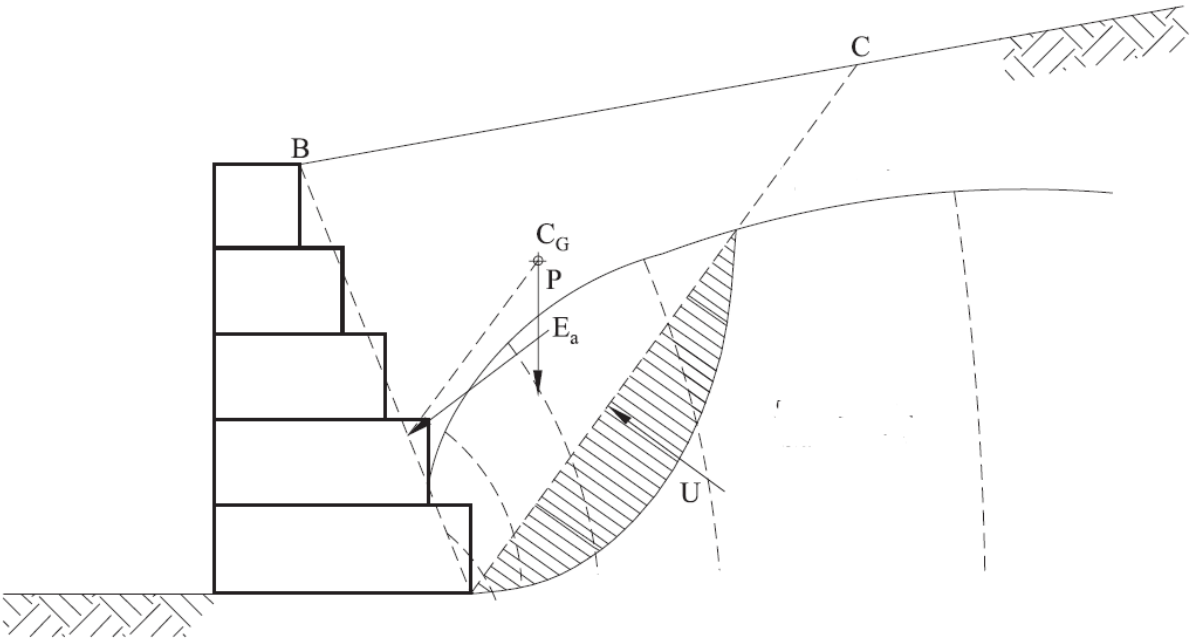


Figure 3 – Porepressure force “U”

2.2.3 Passive Thrust

After calculating the passive thrust, Gawac Increases this force by multiplying it by the partial factor γ_G – fav, which means that this kind of load is favorable for overall stability of the wall. Equation 5 (for granular soils) and Equation 6 represents how this partial factor is applied:

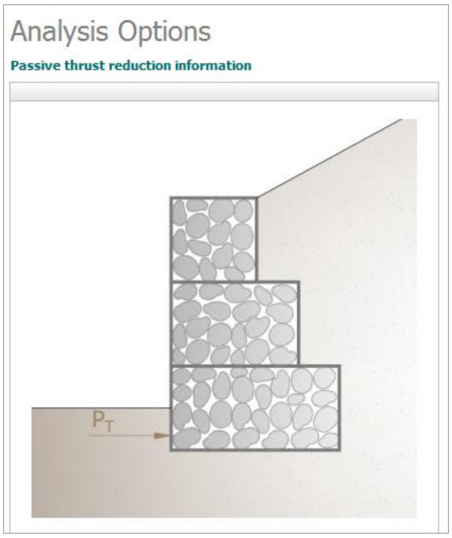


Figure 4 – Passive Thrust - GawacWin

$$E_P = 0.5 \,.\, H^2 . \gamma . kp \quad [5]$$

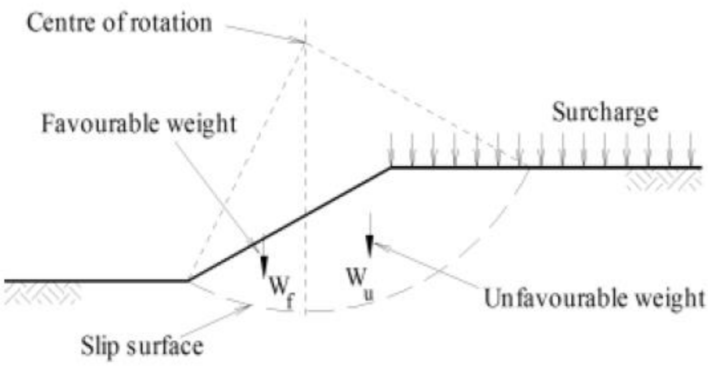
$$E_{P,GD} = E_P \,.\, \gamma_{G,FAV} \quad [6]$$

Where:

E_P Passive Thrust
H Wall embedment
γ Soil unit weight
kp Passive thrust coefficient

2.2.4 Global Stability

The partial factors are also applied on global stability. The right side of the center of rotation of the failure surface corresponds to unfavorable weight, and the left side of center of rotation has a favorable weight (Figure 5a).The partial factors multiplies the weight of each slice (Figure 5b), which may increase or decrease it. The slice weight will be multiplied by γG (Favorable or unfavorable)



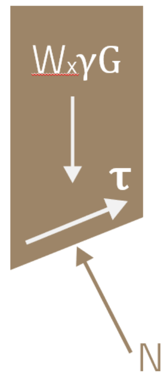


Figure 5 – a) Favorable and Unfavorable Weight b) Partial factor applied on each slice

At the end, the formula to obtain the safety factor against global failure, will be obtained by equation 7:

$$F = \frac{\dfrac{c.B + N.\tan\varphi}{\cos\alpha + \dfrac{\tan\varphi\ .\sin\alpha}{F}}}{\sum(W\ .\gamma G_{FAV\ OR\ UNFAV}).\sin\varphi} \qquad [7]$$

2.3 LOADS

The loads can be defined as:

γG (Permanent) - Dead Loads (e.g. Revetments, walls, structures above the retaining walls)
γQ (Variable) - Live Loads (e.g. Traffic, equipment during the construction)

Besides that, a load can be favorable or unfavorable for the analysis.

The user can choose the load class in the menu Load input (Figure 7).

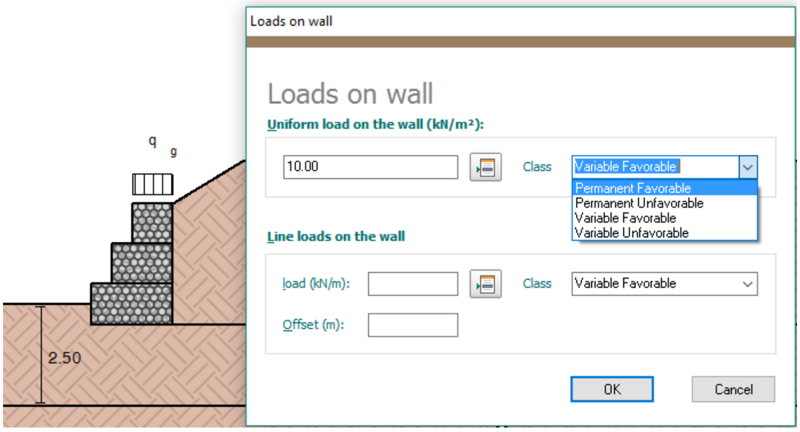


Figure 6 – Choosing class of load on GawacWin

The sections 2.3.1 and 2.3.2 contain examples based on partial factors of Eurocode 7, design approach 1: A1+M1+R1, where the partial factors are presented on Figure 7:

Coefficient of shearing resistence (tan ϕ´)	γϕ´	1.00
Effective cohesion (c´)	γc´	1.00
Undrained shear strength (cu)	γcu	1.00
Permanent action (G) Unfavourable	γG;unfav	1.35
Permanent action (G) Favourable	γG;fav	1.00
Variable action (Q) Unfavourable	γQ;unfav	1.50
Variable action (Q) Favourable	γQ;fav	0.00
Bearing resistance (Rv)	γRv	1.00
Sliding resistence (Rh)	γRh	1.00
Overturning resistance (Rm)	γRm	1.00
Earth internal resistance (Re, internal)	γRe; internal	1.00
Earth overall resistance (Re, overall)	γRe; overall	1.00

Figure 7 – Partial Factors

2.3.1 Favorable Loads

A favorable load (γ_G – fav) is a type of action that somehow can be favorable in one or more analysis.

Considering a load above wall, Qg with 20 kN/m² (Figure 8). The soil properties are presented on table 2.

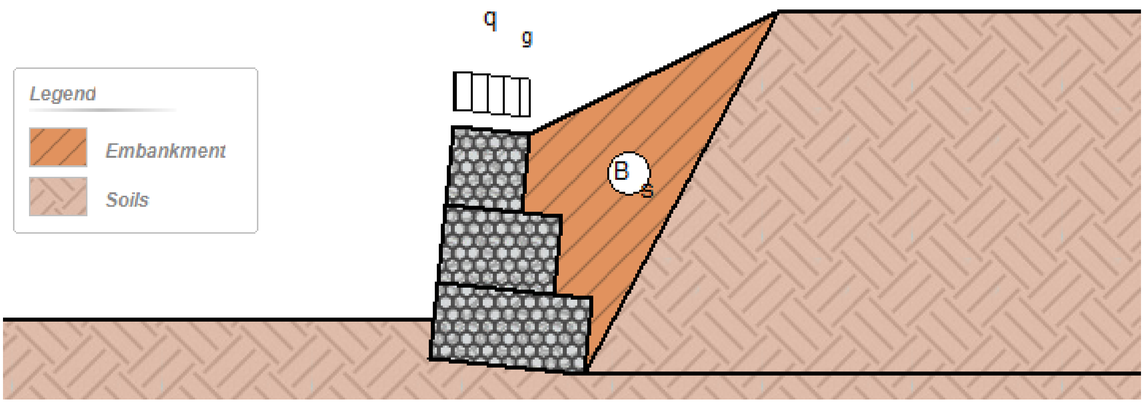


Figure 8 – Load above wall

Table 2 – Soil Properties

Description	Embankment	Soils	Unit
Unit Weight	18	18	kN/m³
Cohesion	0	5	kN/m²
Friction Angle	30	30	°

After choosing the class of load in GawacWin, the software will multiply the load by the factor (γ_Q – fav) or (γ_G – fav):

$$Q_D = Q_K \cdot \gamma_{G,FAV} \quad [8]$$

$$Q_D = Q_K \cdot \gamma_{Q,FAV} \quad [9]$$

Where:

Q_K Characteristic Load (Load input)
Q_D Design Load
γG_{FAV} Partial Factor - Load Permanent Favorable
γQ_{FAV} Partial Factor - Load Variable Favorable

Whenever choosing the class Permanent Favorable, the software will apply the partial factor γ_G – fav. Otherwise, if the class is Permanent Variable, the software will apply the factor γ_Q – fav.

Table 3 shows the difference between the two conditions in an example presented on Figure 9.

Table 3 – Results Comparison

Description	Permanent Favorable	Variable Favorable	Unit
Normal Force on base	140.17	120.28	kN/m
Horizontal Resistance force	113.17	99.60	kN/m
Horizontal Active force	43.07	43.07	kN/m
Sliding Check [FS]	2.63	2.31	-
Restoring Moment	161.91	145.70	kN.m/m
Overturning Moment	39.09	39.09	kN.m/m
Overturning Check [FS]	3.99	3.51	-
Normal Stress on outer border	13.51	87.09	kN/m²
Normal Stress on inner border	5.38	36.41	kN/m²
Allowable stress on foundation	552.02	534.48	kN/m²
FoS – Global Stability	1.86	1.81	-

2.3.2 Unfavorable Loads

An unfavorable load is related to some action which is unfavorable for one or more analysis.

For example, a load above backfill, Q2.

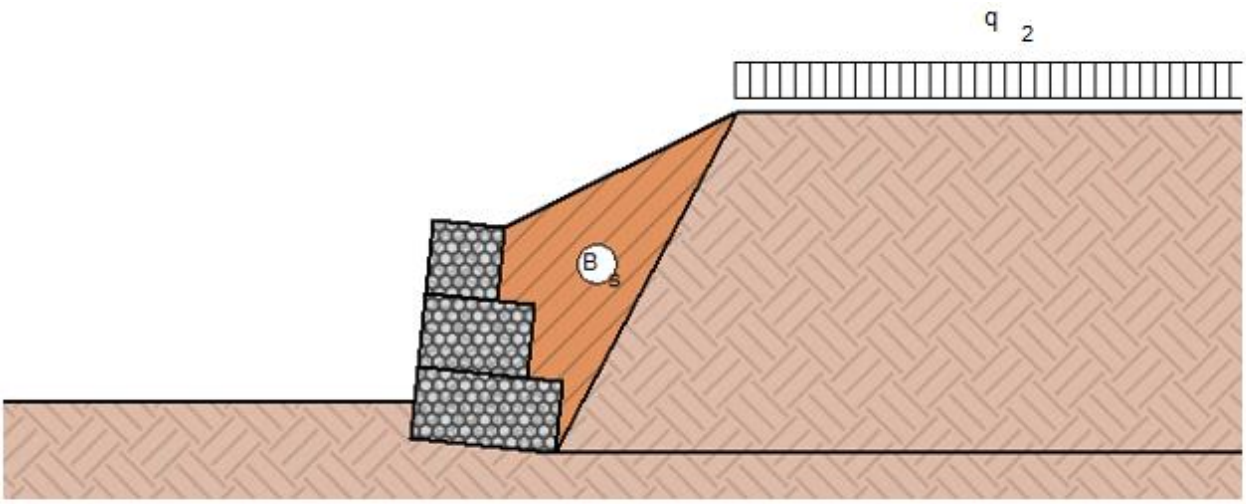


Figure 9 – Load Above Backfill – Q2

After choosing the load class in Gawacwin, the software will multiply the load by the factor (γ_Q – unfav) or (γ_G – fav):

$$Q_D = Q_K \cdot \gamma_{G,UNFAV} \quad [10]$$

$$Q_D = Q_K \cdot \gamma_{Q,UNFAV} \quad [11]$$

Where:

Q_K Characteristic Load (Load input)
Q_D Design Load
γG_{FAV} Partial Factor - Load Permanent Unfavorable
γQ_{FAV} Partial Factor - Load Variable Unfavorable

Table 4 shows the difference between the two conditions in an example presented on Figure 9.

Table 4 – Results Comparison

Description	Permanent Favorable	Variable Favorable	Unit
Normal Force on base	132.53	125.60	kN/m
Horizontal Resistance force	106.67	102.67	kN/m
Horizontal Active force	54.51	48.04	kN/m
Sliding Check [FS]	1.96	2.14	-
Restoring Moment	163.16	153.53	kN.m/m
Overturning Moment	59.37	46.57	kN.m/m
Overturning Check [FS]	2.74	3.35	-
Normal Stress on outer border	115.81	97.20	kN/m²
Normal Stress on inner border	19.91	31.62	kN/m²
Allowable stress on foundation	515.74	525.85	kN/m²
FoS – Global Stability	1.51	1.67	-

2.4 Resistance Partial Factors

When calculating a gabion wall with normative, partial factors are also applied in resistance forces for each calculation. For sliding check, the partial factor γ_{RH} is applied in the sliding resistance (Equation 12)

$$FoS_{Sliding} = \frac{\left(\frac{\text{Tangential Resistance Force}}{\gamma RH}\right)}{\text{Tangential Acting Force}} \quad [12]$$

For overturning check, the partial factor γ_{RM} is applied in the restoring moment resistance (Equation 13).

$$FoS_{Overturning} = \frac{\left(\frac{\text{Restoring Moment}}{\gamma RM}\right)}{\text{Overturning Moment}} \quad [13]$$

For foundation check, the partial factor γ_{Rv} is applied in the bearing resistance (Equation 14).

$$FoS_{Foundation} = \frac{\left(\frac{\text{Ultimate Allowable Bearing Capacity}}{\gamma RV}\right)}{\text{Maximum Foundation Pressure}} \quad [14]$$

For global stability, the partial factor $\gamma_{RE,OVERALL}$ is applied in the resistance forces (Equation 15).

$$FoS_{Global\ Stability} = \frac{\left(\frac{\sum \text{Resistant Forces}}{\gamma RE,\text{overall}}\right)}{\sum \text{Acting Forces}} \quad [15]$$

For internal stability, the partial factor $\gamma_{RE,INTERNAL}$ is applied in the resistance forces (Equation 16). When analyzing gabion structure, it's verified the failure by shear or compression (See next chapter).

$$FoS_{Internal\ Stability} = \frac{\left(\frac{\sum \text{Resistant Forces}}{\gamma RE,\text{internal}}\right)}{\sum \text{Acting Forces}} \quad [16]$$

3. EXAMPLE – NORMATIVE HAND CALCULATION

Gabion filling material | Specific weight: γ_g = 25 kN/m³

Gabion porosity | n= 35%

Reduction due the presence of the geotextile | 5%

Wall batter | β = 0°

Load above wall – Q1 | Permanent Favorable | 10 KPa

Load above backfill – Q2 | Variable Unfavorable | 10 KPa

Foundation allowable Bearing Capacity | 170 KPa

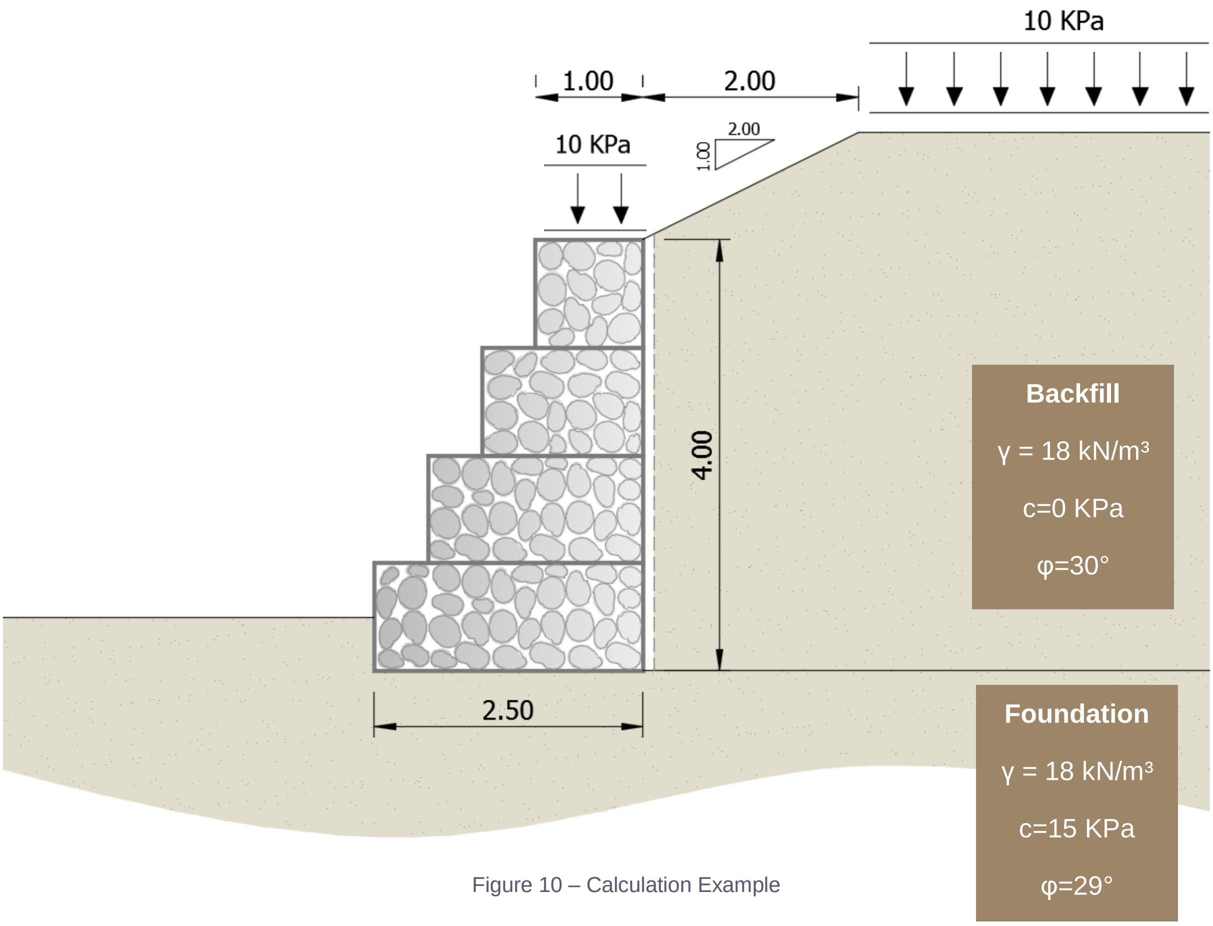


Figure 10 – Calculation Example

Normative: Eurocode 7 EN 1997-1 (EU)

Design Approach: DESIGN APPROACH 1: A2+M2+R1

The partial factors of Design Approach 1: A2+M2+R1 are:

Table 5 – Partial Factors – Design Approach 1

Partial Factor	Related to	Value
$\gamma\varphi$	Friction angle	1.25
γc	Cohesion	1.25
$\gamma c'$	Undrained Shear Strength	1.40
$\gamma G_{,unfav}$	Permanent Action (G) unfavorable	1.00
$\gamma G_{,fav}$	Permanent Action (G) favorable	1.00
$\gamma Q_{,unfav}$	Variable Action (G) unfavorable	1.30
$\gamma Q_{,fav}$	Variable Action (G) favorable	1.00
γRv	Bearing Resistance	1.00
γRh	Sliding Resistance	1.00
γRm	Overturning Resistance	1.00
$\gamma Re_{,intShear}$	Earth internal shear resistance	1.00
$\gamma Re_{,intComp}$	Earth internal compression resistance	1.00
$\gamma Re_{,Overall}$	Earth overall resistance	1.00
$\gamma G_{,wall}$	Wall weight	1.00
$\gamma Q_{,water}$	Water thrust	1.00

Partial Factor	Related to	Value
$\gamma\varphi$ ov	Friction angle | Overturning Analysis	1.25
γc ov	Cohesion | Overturning Analysis	1.25
$\gamma c'$ ov	Undrained Shear Strength | Overturning Analysis	1.40
$\gamma G_{,unfav}$ OV	Permanent Action (G) unfavorable | Overturning Analysis	1.10
$\gamma G_{,fav}$ OV	Permanent Action (G) favorable | Overturning Analysis	0.90
$\gamma Q_{,unfav}$ OV	Variable Action (G) unfavorable | Overturning Analysis	1.50
$\gamma Q_{,fav}$ OV	Variable Action (G) favorable | Overturning Analysis	0.00
γRv ov	Bearing Resistance | Overturning Analysis	1.00
γRh ov	Sliding Resistance | Overturning Analysis	1.00
γRm ov	Overturning Resistance | Overturning Analysis	1.00
$\gamma Re_{,intShear}$ OV	Earth internal shear resistance | Overturning Analysis	1.00
$\gamma Re_{,intComp}$ OV	Earth internal compression resistance | Overturning Analysis	1.00
$\gamma Re_{,Overall}$ OV	Earth overall resistance | Overturning Analysis	1.00
$\gamma G_{,wall}$ OV	Wall weight | Overturning Analysis	1.00
$\gamma Q_{,water}$ OV	Water thrust | Overturning Analysis	1.00

The partial factors related material properties, can be calculated applying the following equations:

$$\varphi_d = \operatorname{atan}\left(\frac{\tan(\varphi)}{\gamma\varphi}\right) \quad [17]$$

$$c_d = \frac{c}{\gamma c} \quad [18]$$

Applying the equations 17 and 18:

Table 6 – Strengh parameters

Soil	Cohesion (c)	Partial Factor	Cohesion (c_d)	Friction Angle (φ)	Partial Factor	Friction Angle(φ_d)
Foundation	15 kPa	1.25	12 kPa	29°	1.25	23.92
Backfill	0 kPa	1.25	0 kPa	30°	1.25	24.79

3.1 ACTIVE THRUST CALCULATION

a) Calculation of active thrust

The calculation of active thrust can be calculated by equilibrium limit method.

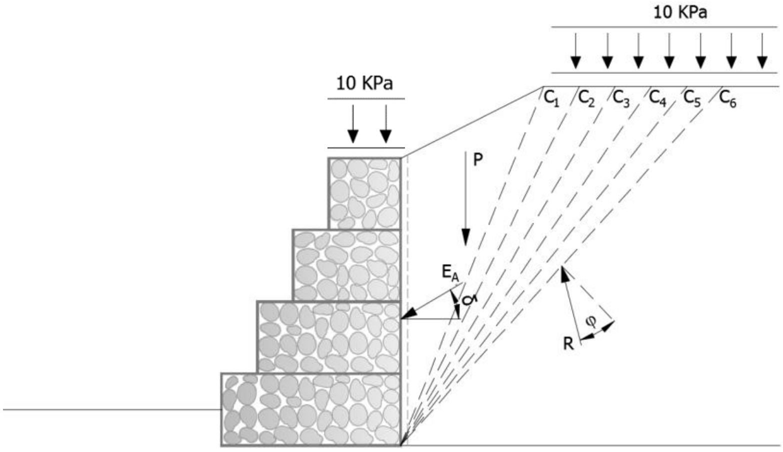


Figure 11 – Wedges of Limit equilibrium method

The active thrust due the soil will give by:

$$Eas = \frac{P.\sin(\rho - \varphi_d)}{\sin(\pi - \alpha - \rho + \varphi_d + \delta)} \quad [19]$$

$$Eas = Eas\ \gamma_{G,UNFAV}$$

The active thrust due the load will give by:

$$Eaq = \frac{Q.\sin(\rho - \varphi_d)}{\sin(\pi - \alpha - \rho + \varphi_d + \delta)} \quad [20]$$

Where “Q” is given by the following equation:

$$Q = [Q2\,.\,(\gamma Q, \text{unfav})]\,.\,L \quad [21]$$

Where L is the distance that the load is applied, this value is variable for each wedge.

The angle “α” is 90°.

Table 7 – Active Thrust Results

Wedge	ρ [°]	D [m]	EAs [kN/m]	EAq [kN/m]	EA [kN/m]
1.00	68.20	2.00	52.60	0.00	52.60
2.00	63.43	2.50	61.12	4.20	65.33
3.00	59.04	3.00	67.00	7.44	74.45
4.00	55.01	3.50	70.68	9.88	80.56
5.00	51.34	4.00	72.51	11.64	84.14
6.00	**48.01**	**4.50**	**72.75**	**12.81**	**85.56**
7.00	45.00	5.00	71.63	13.50	85.12

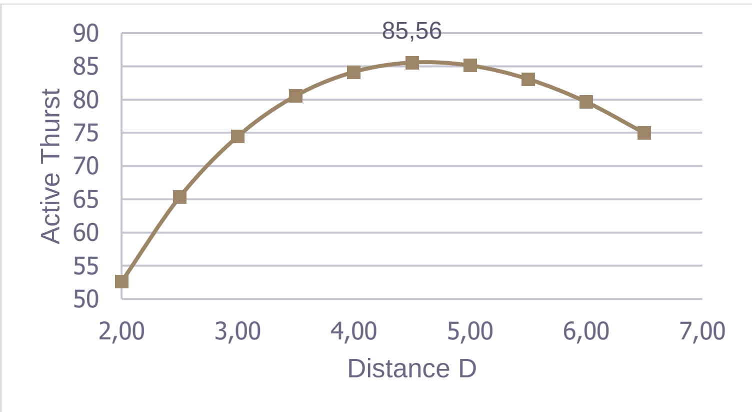


Figure 12 – Active Thrust Results

The maximum earth pressure is **85.56 kN/m**. The angle **ρ** is **48.01°** and the distance D is 4.50m.

The application point of active thrust of soil can be obtained by the center of gravity of the critical wedge. After that, a parallel line to the angle ρ is project until the wall interface (Figure 13)

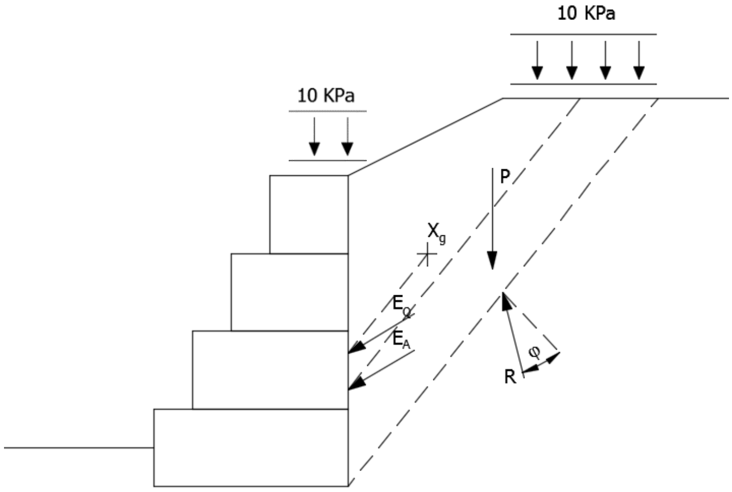


Figure 13 – Critical wedge

b) Center of gravity | Wedge

The center of gravity can be determined by dividing the wedge in two parts (I and II):

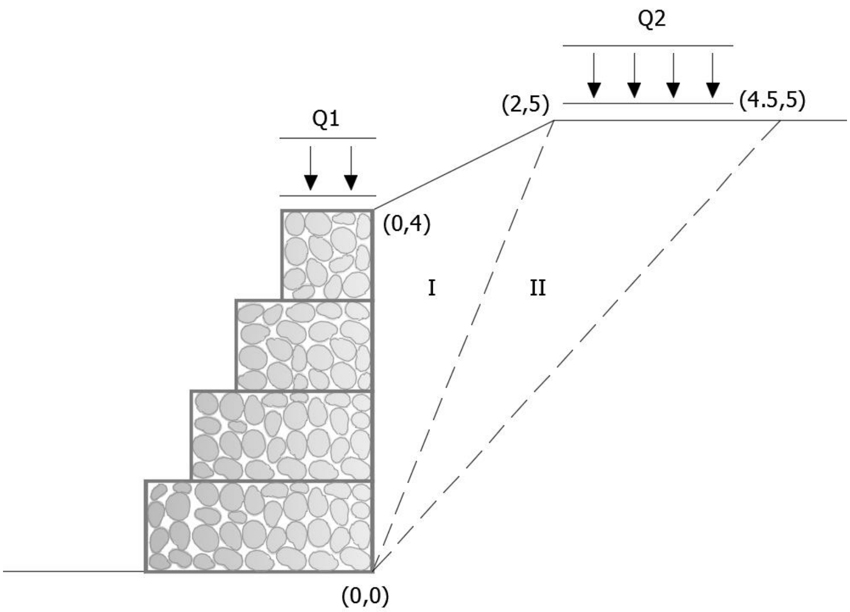


Figure 14 – Example 4 – Calculation of center of gravity

Then, the center of gravity of the wedge is:

$$Xg = 1.58m$$

$$Yg = 3.20m$$

c) Point of active thrust (Y) - Soil

Then, the application point of active thrust due the soil can be determined by projecting the center of gravity on the interface of the wall (Figure 15):

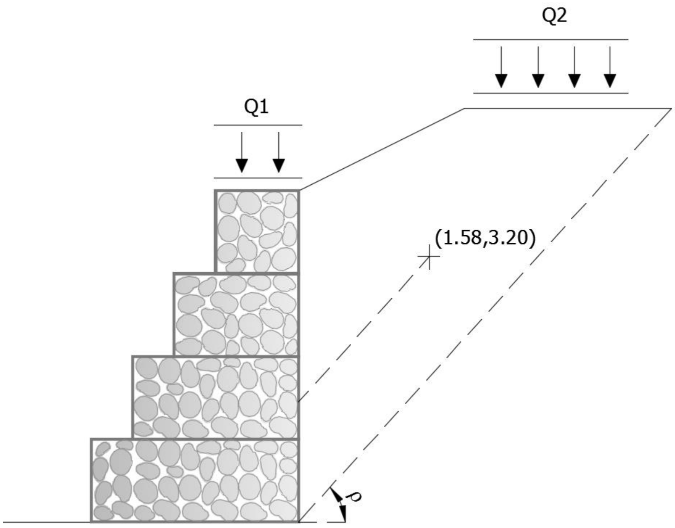


Figure 15 - Point of active thrust - Soil

$$YEA = Yg - Xg.\tan\rho \qquad [22]$$

$$YEA = 1.46m$$

d) Point of active thrust (Y) - Load

The application point of active thrust caused by the load can be determined by projecting a parallel line of the angle ρ of the center of the load (Figure 16).

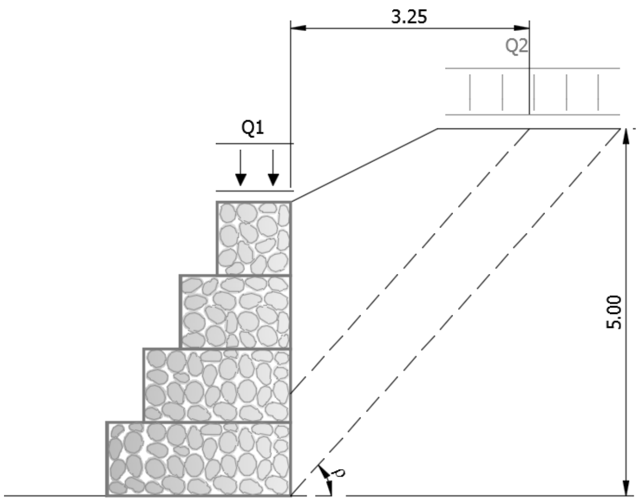


Figure 16 – Point of application – Load Active thrust

$$\mathrm{YEAq} = 5 - 3.25\,.\tan(48.01)$$

$$\mathrm{YEAq} = 1.39\mathrm{m}$$

e) Point of application – Active Thrust

The total active thrust can be calculated by the weighted average of the two parcels (Eas and Eaq)

E_{AS}	72.75 kN/m	YEas	1.46m
E_{AQ}	12.81 kN/m	YEaq	1.39m

Then, the point of application of active thrust will be given by the weighted average:

$$\mathrm{YEa} = \frac{72.75 \text{ x } 1.46 + 12.81 \text{ x } 1.39}{72.75 + 12.81}$$

$$\mathrm{YEa} = 1.45\mathrm{m}$$

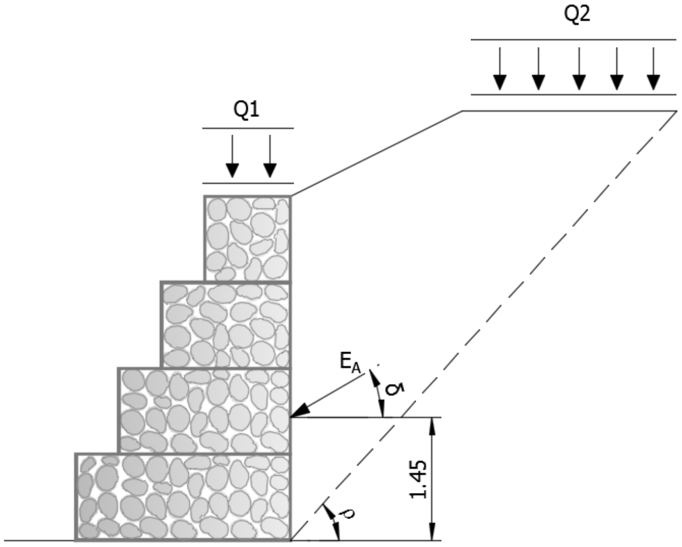


Figure 17 – Point of application | Active Thrust

4.3 SLIDING CHECK

a) Calculation of normal force on the base:

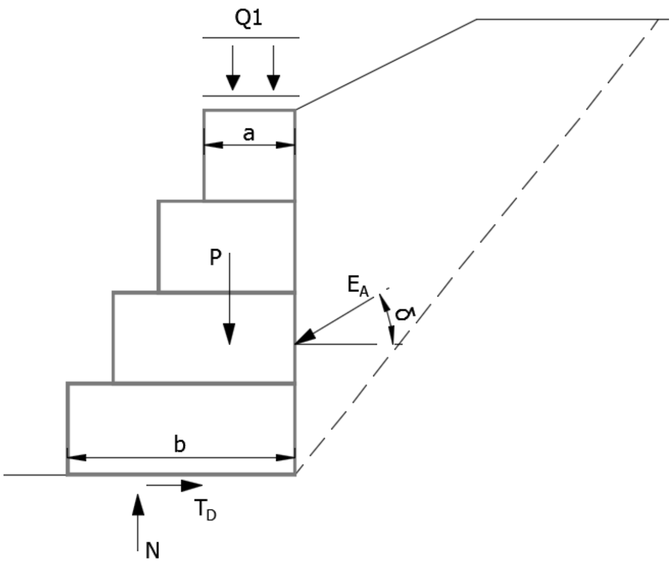


Figure 18 – Forces on base

The Normal force will be given by:

$$N = P\ x\ \gamma Gwall\ + \mathrm{Ea.\,sen}(\delta) + Q1\ .\ \gamma Q,\mathrm{fav}\ .\ a$$

The force “P” is given by:

$$P = Ag \,.\, \gamma g \,(1 - n) \qquad [24]$$

Where “γg” is the specific weight of the gabions (25kN/m³), “n” is the gabion porosity (n=35%) and Ag is the area of the cross section.

$$Ag = (1\,x1 + 1.5\,x\,1 + 2x1 + 2.5x1)$$

$$P = 7.0\ m^2 . 25kN/m^3\ (1 - 0.35)$$

Substituting the values on equation 23, the force N will be:

$$N = 113.75\ x\ \gamma G wall + 85.56 . sen(23.55) + 10\,.\,1\,.\,1$$

$$N = 157.93\ kN$$

b) Calculation of tangential force on base:

The force “Td” will be given by:

$$Td = N\,.\,\tan\varphi_d{}^* + c_d . B \qquad [25]$$

Where c_d is the soil cohesion of foundation. By reason of security, this value can be divided by 2.

$$Td = 157.93\,.\,\tan(23.92) + \frac{12}{2} . 2.5$$

$$Td = 85.05\ \ kN/m$$

The Force Td is divided by the factor γ_{RH}, which is equal to 1.00 on this condition, then:

$$Td = \frac{85.05}{\gamma RH}$$

$$Td = 85.05\ \ kN/m$$

Then, the safety factor will be:

$$FS_{slid} = \frac{Td}{Ea.\cos(\delta d)} \qquad [26]$$

$$FS_{slid} = \frac{85.05}{85.56\,.\cos(23.55)}$$

$$\mathbf{FS_{slid} = 1.08}$$

4.4 OVERTURNING CHECK (EQU)

a) Calculation of active moment:

To check the overturning analysis, all the calculations concern active thrust must be done with the partial factors Υ_{OV}.

The active moment will be given by:

$$ME_{AH} = EA_{OV}\,.\cos(\delta).yEA \qquad [27]$$

$$ME_{AH} = 94.81\,.\cos(23.55).1.50$$

$$ME_{AH} = 130.37\frac{kN}{m}.m$$

b) Calculation of restoring moment:

Calculation of the center of gravity of gabion wall:

$$X'g = \frac{\sum Ai\,.\,xgi}{\sum A} \qquad [28]$$

$$X'g = \frac{(2.5x\ 1.25) + (2\ x\ 1.5) + (1.5\,.\,1.75) + (1\,.\,2)}{(2.5 + 2 + 1.5 + 1)}$$

$$X'g = 1.53m$$

The resistant moment of the wall will be:

$$Mp = X'g \, . \, P \, . \, \gamma GwallOV \qquad [29]$$

$$Mp = 1.53 \, . \, 113{,}75 \, . \, 1$$

$$Mp = 174{,}03 \; kN.m$$

The resistant moment due the vertical parcel of active thrust will be:

$$M_{EAV} = Ea \, . \, sen(\delta) \; . \, XEa$$

$$M_{EAV} = 94.81 \, . \, sin(23.5) \; . \, 2.5$$

$$M_{EAV} = 94.51 \; \frac{kN}{m} . \, m$$

The resistant moment due the load on top of the wall will be:

$$M_{Q1} = (Q1. \, a \, . \, \gamma Q, favOV). \, x \qquad [30]$$

Where “x” is the distance between the point of rotation and the center of gravity of the load Q1.

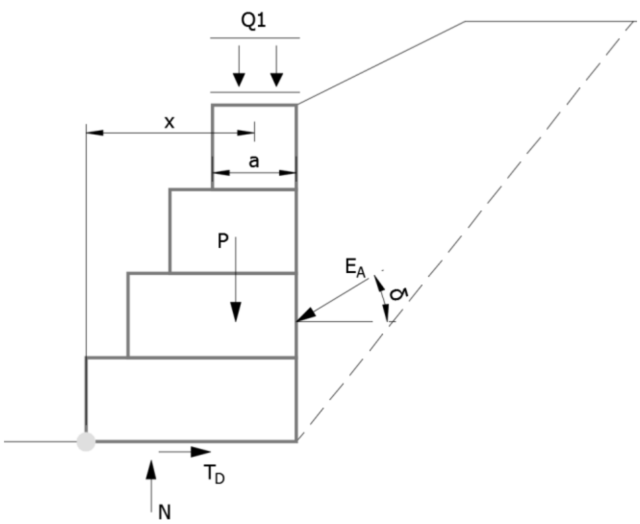


Figure 19 – Resistance moment due the load Q1

$$M_{Q1} = (10 \,.\, 1 \,.\, 0.90) \,.\, 2$$

$$M_{Q1} = 18 \, \frac{kN}{m} . m$$

The total restoring moment will be given by:

$$M_R = Mp + M_{EAV} + M_{Q1}$$ [31]

$$M_R = 174.03 + 94.51 + 18$$

The total restoring moment is divided by γ_{RH}, which is 1.00.

$$M_R = \frac{286.54}{\gamma RH}$$

$$M_R = 286.54 \frac{kN}{m} . m$$

$$FSovr = \frac{M_R}{ME_{AH}}$$ [32]

$$FSovr = \frac{286.54}{130.37}$$

$$\mathbf{FSovr = 2.20}$$

4.5 FOUNDATION CHECK

a) Eccentricity calculation

The distance "d", between the base point and the point of application of force "N".

$$d = \frac{Mp + MEa + M_{Q1} - MEah}{N}$$ [33]

Where M_P and M_{EA} are calculated considering the standard partial factors, that's why these values are different than the EQU analysis.

$$d = \frac{174.03 + 85.44 + 20 - 117.62}{157.93}$$

$$d = 1.02m$$

Eccentricity calculation:

$$e = \frac{B}{2} - d = \frac{2.5}{2} - 1.02 = 0.22m$$

Eccentricity condition:

$$0.22 < \frac{2.5}{6}$$

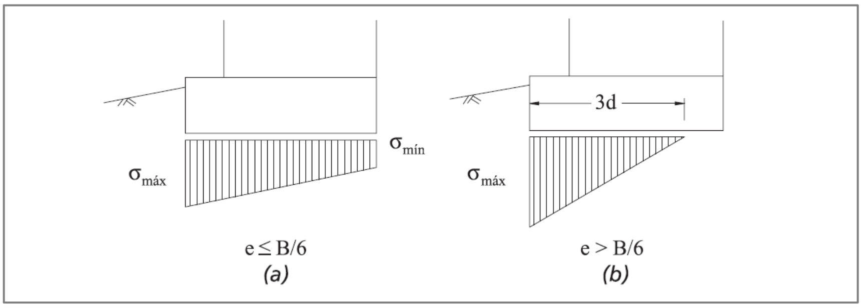


Figure 20 – Pression distribution on base

Then:

$$qmax = \frac{N}{B}.(1 + 6.\frac{e}{B})$$

$$qmax = \frac{157.93}{2.5}.(1 + 6.\frac{0.22}{2.5})$$

$$qmax = 96.53 \ \frac{kN}{m^2} < qadm = 170 \ kN/m^2 \therefore \textbf{Ok}$$

$$qmin = \frac{N}{B}.(1 - 6.\frac{e}{B})$$

$$qmin = \frac{157.93}{2.5}.(1 - 6.\frac{0.22}{2.5})$$

MACCAFERRI

$$q_{mín} = 29.81.\frac{kN}{m^2} < q_{adm} = 170\ kN/m^2 \therefore \textbf{Ok}$$

4.6 INTERNAL STABILITY | ULS

The internal stability can be calculated using ULS analysis. The calc procedure is:

- Find the maximum active thrust on each layer.
- Calculation of the horizontal active force on each layer.
- Compare the active force with the maximum allowable shear stress on gabion.
- Calculation of the normal force on each layer.
- Compare the Normal Stress with the maximum allowable stress.

Figure 21 presents the results on each layer.

Internal stability

Layer	H [m]	N [kN/m]	T [kN/m]	M [kN/m x m]	τ Max [kN/m]	τ All [kN/m²]	τ FoS	σ Max [kN/m²]	σ All [kN/m²]	σ FoS
1	1.00	29.55	7.56	13.42	7.56	49.36	**6.53**	32.53	518.29	**15.93**
2	2.00	61.25	24.38	41.62	16.25	60.64	**3.73**	45.07	518.29	**11.50**
3	3.00	104.14	48.21	89.43	24.11	71.88	**2.98**	60.63	518.29	**8.55**

Figure 21 – Internal Stability results

The values of σ_{ALL} and τ_{ALL} are divided by the partial factor $\gamma_{RE,INTERNAL}$.

4.7 GLOBAL STABILITY

The global stability can be verified using computers due the iteration equations and curves. The software GawacWin provides this analysis.

Global Stability | Bishop

Center of the arch referred to X axis [m]	0.62
Center of the arch referred to Y axis [m]	6.24
Overall Stability Check	1.26

Figure 22 – Global Stability Analysis

APPENDIX I – GAWAC 3.0 REPORT

Project Information

Title	Client	Description
Number	Designer	
		Comments

Input

Eurocode 7 EN 1997-1 (EU) - DESIGN APPROACH 1: A2+M2+R1

Wall data

Wall batter [°]	0.00
Rockfill unit weight [kN/m³]	25.00
Porosity of gabions [%]	35.00
Geotextile in the backfill	Yes
Friction reduction [%]	5.00
Geotextile on the base	No
Friction reduction [%]	0.00

Backfill soil data

Inclination of Stretch 1 [°]	26.57
Length of stretch 1 [m]	2.00
Inclination of Stretch 2 [°]	0.00
Soil unit weight [kN/m³]	18.00
Soil friction angle [°]	30.00
Soil cohesion [kN/m²]	0.00

Layer	Initial height [m]	Incl. angle deg	Unit weight [kN/m³]	Cohesion [kN/m²]	Friction angle [deg]

Foundation data

Top surface height [m]	0.00
Top surface init. length [m]	
Top surface incl. angle [°]	0.00
Soil unit weight [kN/m³]	19.00
Soil friction angle [°]	29.00
Soil cohesion [kN/m²]	15.00
Foundation allowable pressure [kN/m²]	170.00
Water table height [m]	

Layer	Depth [m]	Unit weight [kN/m³]	Cohesion [kN/m²]	Friction angle [deg]
1	2.00	18.00	20.00	30.00

Loads data

Distributed loads on backfill			
First stretch [kN/m²]	Variable Unfavourable	q1	
Second stretch [kN/m²]	Permanent Favourable	q2	10.00
Distributed loads on wall			
Load [kN/m²]	Permanent Favourable		10.00
Line loads on backfill			
Load 1 [kN/m]	Variable Unfavourable		
Distance from wall face [m]			
Load 2 [kN/m]	Variable Unfavourable		
Distance from wall face [m]			
Load 3 [kN/m]	Variable Unfavourable		
Distance from wall face [m]			
Line load on wall			
Load [kN/m]	Variable Favourable		
Distance from wall face [m]			0.00

Phreatic surface data

Initial height [m]	0.00
Inclination of the 1st stretch [°]	0.00
Length of the 1st stretch [m]	0.00
Inclination of the 2nd stretch [°]	0.00
Length of the 2nd stretch [m]	0.00

Seismic action data

Horizontal coefficient	
Vertical coefficient	

Product

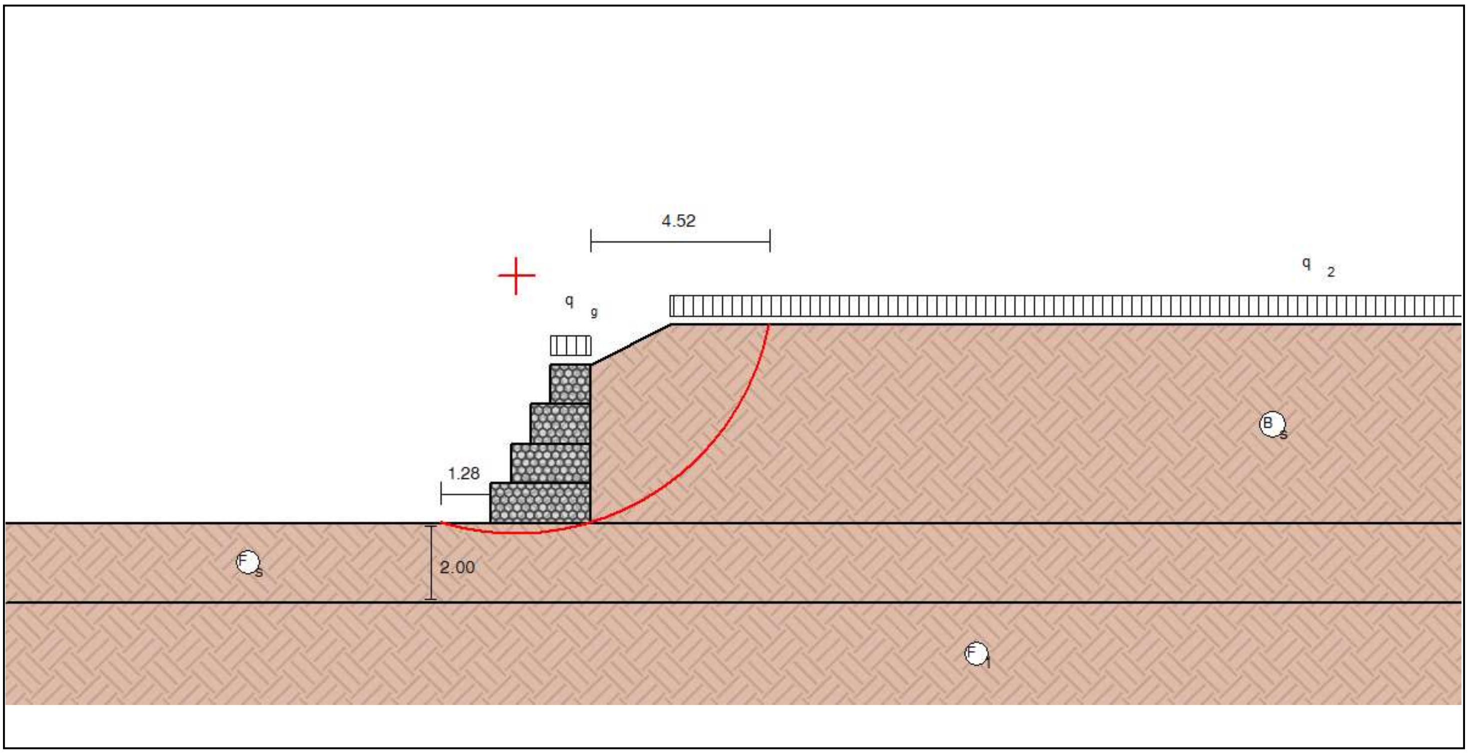
Gabion type:	POLIMAC™ 60/528
GSC 0.5 m	1246
GSC 1.0 m	623
Ambient	Low Aggressive

Results

Eurocode 7 EN 1997-1 (EU) - DESIGN APPROACH 1: A2+M2+R1

ULS Ultimate Limit State

Wall Design



Stability Analysis Results

Active and Passive Thrust

Active Thrust [kN/m]	82.58
Point of application ref. to X axis [m]	2.50
Point of application ref. to Y axis [m]	1.48
Direction of the thrust ref. to X axis [deg]	23.55
Passive Thrust [kN/m]	0.00
Point of application ref. to X axis [m]	0.00
Point of application ref. to Y axis [m]	0.00
Direction of the thrust ref. to X axis [deg]	0.00

Sliding

Normal force on the base [kN/m]	156.75
Point of application ref. to X axis [m]	1.05
Point of application ref. to Y axis [m]	0.00
Tangential active force [kN/m]	75.70
Tangential resistance force [kN/m]	84.51
Sliding check	1.12

Overturning

Overturning Moment [kN/m x m]	123.53
Restoring Moment [kN/m x m]	287.76
Overturning check	2.33

Stresses Acting on Foundation

Eccentricity	0.20
Normal stress on outer border [kN/m²]	92.33
Normal stress on inner border [kN/m²]	33.07
Allowable stress on foundation [kN/m²]	170.00
Stress on foundation (Toe) check	1.84
Stress on foundation (Heel) check	5.14

Global Stability | Bishop

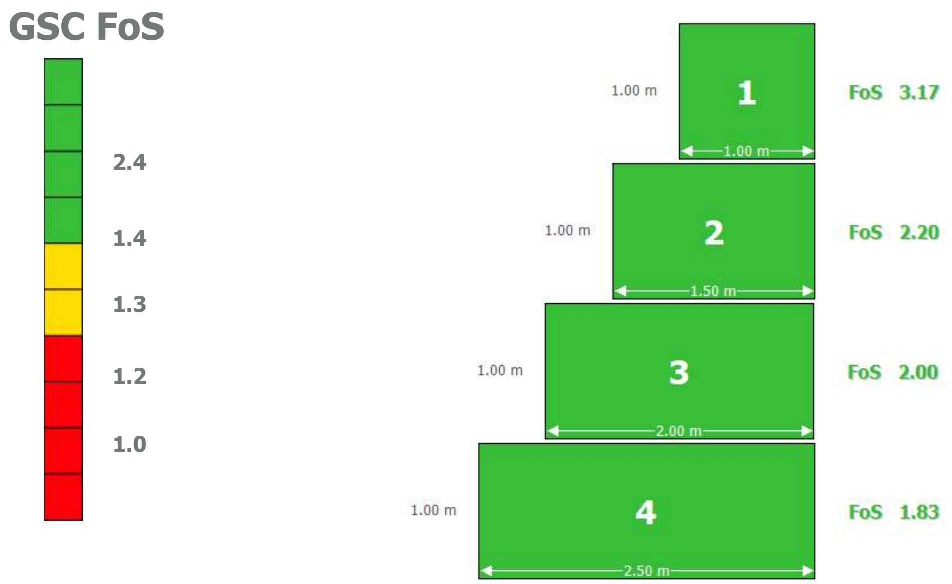
Center of the arch referred to X axis [m]	0.62
Center of the arch referred to Y axis [m]	6.24
Overall Stability Check	1.28

Results

Eurocode 7 EN 1997-1 (EU) - DESIGN APPROACH 1: A2+M2+R1

SLS Seviceability Limit State

Gabion Serviceability Coefficient



ULS Ultimate Limit State

External stability

Overturning check	Sliding check	Stress on foundation (Toe) Check	Stress on foundation (Heel) Check
FoS 2.33	FoS 1.12	FoS 1.84	FoS 5.14

Global

Global Stability Check
FoS 1.28

Internal stability

Layer	H [m]	N [kN/m]	T [kN/m]	M [kN/m x m]	τ_{Max} [kN/m]	τ_{All} [kN/m²]	τ FoS	σ_{Max} [kN/m²]	σ_{All} [kN/m²]	σ FoS
1	1.00	29.37	7.17	13.45	7.17	49.18	**6.86**	32.09	518.29	**16.15**
2	2.00	60.79	23.32	42.00	15.54	60.34	**3.88**	43.99	518.29	**11.78**
3	3.00	103.33	46.36	90.62	23.18	71.48	**3.08**	58.91	518.29	**8.80**

Normative

Eurocode 7 EN 1997-1 (EU)

DESIGN APPROACH 1: A2+M2+R1
No Seismic Condition

Partial Factors

			Overturning
Coefficient of shearing resistance	$\gamma\phi'$	1.25	1.25
Effective cohesion	$\gamma c'$	1.25	1.25
Undrained shear strength	γcu	1.40	1.40
Permanent action (G) Unfavourable	$\gamma G;unfav$	1.00	1.10
Permanent action (G) Favourable	$\gamma G;fav$	1.00	0.90
Variable action (Q) Unfavourable	$\gamma Q;unfav$	1.30	1.50
Variable action (Q) Favourable	$\gamma Q;fav$	1.00	1.00
Bearing resistance	γRv	1.00	1.00
Sliding resistence	γRh	1.00	1.00
Overturning resistance	γRm	1.00	1.00
Earth internal resistance shear	γRe; intShear	1.00	1.00
Earth internal resistance compression	γRe; intComp	1.00	1.00
Earth overall resistance	γRe; overall	1.00	1.00
Gabion wall height	γ G; Wall	1.00	1.00
Water Thrust	γ Water	1.00	1.10