SLOPE STABILITY ANALYSIS MANUAL - THEORY

DeepEX software program (Version 2018)
SnailPlus software program (Version 2018)

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A. INTRODUCTION

Aims of Slope Stability analysis

In most cases, the primary purpose of slope stability analysis is to contribute to the safe and economic design of excavations, embankments, earth dams, landfills, and soil heaps. Slope stability evaluations are concerned with identifying critical geological, material, environmental, and economic parameters that will affect the project, as well as understanding the nature, magnitude, and frequency of potential slope problems. When dealing with slopes in general and slope stability analysis in particular, previous geological and geotechnical experience in an area is valuable.

The aims of slope stability analyses are

(i) To understand the development and form of natural slopes and the processes responsible for different natural features.

(ii) To assess the stability of slopes under short-term (often during construction) and long-term conditions.

(iii) To assess the possibility of landslides involving natural or existing engineered slopes.

(iv) To analyze landslides, understand failure mechanisms, and evaluate the influence of environmental factors.

(v) To enable redesign of failed slopes and planning and design of preventive and remedial measures (when necessary).

(vi) To study the effect of earthquakes on slopes and embankments.

Slope stability analysis must take into account a variety of factors relating to topography, geology, and material properties, often associated to whether the slope was naturally formed or engineered.
B. METHOD OF SLICES

1. SIMPLIFIED BISHOP METHOD

The simplified Bishop method uses the method of slices to discretize the soil mass and determine the FS (Factor of Safety). This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial surface. Since horizontal forces are not considered at each slice, the simplified Bishop method also assumes zero interslice shear forces. Using the notation shown in Figure B.1, the overall moment equilibrium of forces acting on each slice is provided by:

\[
\sum M_0 = \sum_{i=1}^{n} \left[ W(1 - k_u) + U_\beta \cos \beta + Q \cos \delta \right] R \sin \alpha \\
- \sum_{i=1}^{n} \left[ U_\beta \sin \beta + Q \sin \delta \right] (R \cos \alpha - h)
- \sum_{i=1}^{n} [S_m] R + \sum_{i=1}^{n} [k_h W(R \cos \alpha - h_c)] = 0
\]

Where:
- \( R \) = radius of circular failure surface
- \( h \) = average height of the slice
- \( h_c \) = vertical height between center of base slice and centroid of the slice

The above equation may be simplified by dividing throughout by the ratios to get:

\[
\frac{\sum M_0}{R} = \sum_{i=1}^{n} \left[ W(1 - k_u) + U_\beta \cos \beta + Q \cos \delta \right] \sin \alpha \\
- \sum_{i=1}^{n} \left[ U_\beta \sin \beta + Q \sin \delta \right] (\cos \alpha - \frac{h}{R}) \\
- \sum_{i=1}^{n} [S_m] + \sum_{i=1}^{n} [k_h W(\cos \alpha - \frac{h_c}{R})] = 0
\]

Note that the effective normal and pore pressure forces, acting on the base slice, do not affect the moment equilibrium expression since they pass through the center of the circle. Thus, Bishop’s method should not be applied to compute an FS for non-circular surfaces.
If the same FS is assumed for all the slices, the Mohr – Coulomb criterion becomes:

\[
F = \frac{\sum_{i=1}^{n}(C + N' \tan \varphi)}{\sum_{i=1}^{n}A_5 - \sum_{i=1}^{n}A_6 - \sum_{i=1}^{n}A_7}
\]

[EQ. B1.1]

Where \( A_5 = [W(1 - k_u) + U_\beta \cos \beta + Q \cos \delta] R \sin \alpha \)

\( A_6 = [U_\beta \sin \beta + Q \sin \delta] (\cos \alpha - \frac{h}{R}) \)

\( A_7 = k_h W(\cos \alpha - \frac{h}{R}) \)


Next, forces are summed in the vertical direction for each slice, and the effective normal force is calculated as:

\[
N' = \frac{1}{m_a} [W(1 - k_u) - \frac{C \sin \alpha}{F} - U_\alpha \cos \alpha + U_\beta \cos \beta + Q \cos \delta]
\]

[EQ. B1.5]

Where \( m_a \) is:

\[
m_a = \cos \alpha \left(1 - \frac{\tan \alpha \tan \varphi}{F}\right)
\]

[EQ. B1.6]
The equations above list the expressions that are used to calculate the FS for circular surfaces according to the simplified Bishop method.

**Figure B.1: Forces acting on a typical slice**
2. GENERALIZED LIMIT EQUILIBRIUM (GLE) METHOD

The GLE method is an extension of Spencer’s (1973) method, which has been generalized by Chugh (1986). The GLE method adopts a function, $\theta_L = \lambda \cdot f(x_i)$ to assign the interslice force angle on the right-hand side of the slice $i$, as shown in Figure B.1. The function $f(x_i)$ ranges between 0 and 1 and essentially represents the shape of the distribution used to describe the variation of the interslice force angles, as shown in Figure C.1. The adoption of this function satisfies $(n - 1)$ assumptions regarding the interslice force angles and the $\lambda$ value is an additional unknown, which is introduced such that there are $(n - 1)$ unknowns, as discussed earlier. The selected interslice force angle function, $f(x)$, can be set as a constant (i.e., $f(x) = 1.0$) to emulate Spencer’s procedure, or any other shape for a discrete version of a Morgenstern-Price solution.

The adopted formulation uses a discrete form of the continuous function, $f(x)$, to calculate the function at each interslice boundary, using the angles $\theta_L$ and $\theta_R$ for the left and right vertical sides of the slice, as shown in Figure B1.1. Thus for a typical interslice boundary, $\theta_R = \lambda \cdot f(x)$, where $x$ is the horizontal coordinate of the right side of the selected slice with respect to the toe, the interslice force inclination angle can be determined. This distribution is usually implemented with a function that is normalized with respect to the lateral (horizontal) extent of the failure surface. As the interslice force angle for the left side of the first slice (at the toe) and the right side of the last slice (at the crest) is assumed to be zero, this lateral extent is assumed to range between the first and the last interslice boundary.

![Function f(x)](image)

**Figure B.2:** Examples of functions used to describe the variation of interslice force angles.
Force Equilibrium  As Figure B.1 shows, the GLE method assumes that the interslice resultant forces, \( Z_L \) and \( Z_R \), are inclined at \( \theta_L \) and \( \theta_R \) on the left and right side of each slice. These interslice forces are total forces as the hydrostatic component along the interslice boundaries is not considered separately. Interslice hydrostatic forces are considered in the analysis with the software diagram, but are difficult to implement for layered soils and multiple water surfaces in hand calculation. If force equilibrium is considered in a direction parallel to the base of each slice,

\[
S_m + Z_L \cos(a - \theta_L) - Z_R \cos(a - \theta_R) - W(1 - k_u) \sin a - Wk_h \cos a - U_{\beta} \sin(\alpha - \beta) - Q \sin(\alpha - \delta) = 0
\]

[EQ. B2.1]

and if the Mohr-Coulomb strength criterion is adopted, then the mobilized strength is:

\[
S_m = \frac{S_a}{F} = \frac{C}{F} + N' \frac{\tan \varphi_m}{F} = C_m + N' \tan \varphi_m
\]

[EQ. B2.2]

then substituting Equation C.2 to C.1, the following expression is obtained:

\[
N' \tan \varphi_m = Z_R \cos(a - \theta_R) - Z_L \cos(a - \theta_L) + W[(1 - k_u) \sin a + k_h \cos a] - C_m + U_{\beta} \sin(\alpha - \beta) + Q \sin(\alpha - \delta)
\]

[EQ. B2.3]

Next force equilibrium is formulated in a direction normal to the base of the slice:

\[
N' + Z_R \sin(a - \theta_R) - Z_L \sin(a - \theta_L) - W(1 - k_u) \cos a + Wk_h \sin a + U_a - U_{\beta} \cos(\alpha - \beta) - Q \cos(\alpha - \delta)
\]

[EQ. C4]

By substituting Equation B2.4 to B2.3, the following force equilibrium equation is derived:

\[
Z_R = A_8 Z_L [\cos(a - \theta_L) + \sin(a - \theta_L) \tan \varphi_m] + A_8 [W \cos a (1 - k_u)(\tan \varphi_m - \tan a) + C_m - U_a \tan \varphi_m - Wk_h (1 + \tan \varphi_m \tan a) \cos a + U_{\beta}[\cos(\alpha - \beta) \tan \varphi_m - \sin(\alpha - \beta)] + Q[\cos(\alpha - \delta) \tan \varphi_m - \sin(\alpha - \delta)]
\]

[EQ. B2.5]

Where the factor \( A_8 \) is given by
Moment Equilibrium  The conditions for moment equilibrium are satisfied by taking moments of all slice forces about the midpoint of the base of the slice, as shown in Figure B2.2, generating the following expression:

\[
Z_L \cos \theta_L \left[ h_L - \frac{b}{2} \tan a \right] + Z_L \frac{b}{2} \sin \theta_L - Z_R \cos \theta_R \left[ h_R - \frac{b}{2} \tan a \right] + Z_R \frac{b}{2} \sin \theta_R - W k_h h_c \\
+ U_\beta h \sin \beta + Q h \sin \delta = 0
\]

[EQ. B2.7]

Next the above expression is simplified to determine the location of the interslice force, \( h_R \), on the right-hand side of each slice, using

\[
h_R = \frac{Z_L}{Z_R \cos \theta_R} \left[ h_L \cos \theta_L - \frac{b}{2} (\cos \theta_L \tan a + \sin \theta_L) \right] \\
+ \frac{1}{Z_R \cos \theta_R} \left[ h (U_\beta \sin \beta + Q \sin \delta) - W k_h h_c \right] + \frac{b}{2} [\tan \theta_R - \tan a]
\]

[EQ. B2.8]

The GLE procedure uses Equations B2.5 and B2.8 iteratively to satisfy complete moment and force equilibrium for all slices. Once the FS has been determined, the total normal, vertical and shear stresses at the base of each slice should be calculated using

\[
\sigma_n = \frac{1}{b \sec a} \left[ Z_L \sin(a - \theta_L) - Z_R (a - \theta_R) + U_\beta \cos(\alpha - \beta) - U_a + W [(1 - k_u) \cos a \\
+ k_R \sin a] + Q \cos(\alpha - \delta) \right]
\]

[EQ. B2.9]

\[
\sigma_u = \frac{W + Q \cos \delta + U_\beta \cos \beta}{b \sec a}
\]

[EQ. B2.10]

\[
\tau_{base} = C_m + \sigma_n ' \tan \varphi_m
\]

[EQ. B2.11]
Solution Procedure

The GLE solution is computed using the following steps:

(i) Assume an interslice force angle distribution with $\theta_L$ on the first slice and $\theta_R$ on the last slice set to zero.

(ii) Determine the FS that allows Equations C5 and C8 to satisfy force equilibrium such that $Z_R$ for the last slice (at the crest) is equal to the boundary force. This force will be equal to the hydrostatic water force in a water-filled crack at the crest of the slope. If there is no water-filled crack, this boundary force will be zero.

(iii) Retain the calculated interslice forces, $Z_L$ and $Z_R$, that were part of the solution for the FS.

(iv) Using the interslice forces from step (iii), use Equation B2.8 to calculate the magnitude of the interslice forces angles, $\theta_R$, that satisfy moment equilibrium such that $h_R$ for the last slice is zero or equal to the location of the horizontal hydrostatic force in a water-filled crack. These calculations are performed sequentially for each slice, starting with the knowledge that $\theta_L$ and $h_L$ for the first slice will be zero.

(v) Repeat steps (ii) to (iv), until the calculated FS and the interslice force angles are within a tolerable limit.

(vi) Calculate the total normal, vertical and shear stresses at the base of each slice, using Equations B2.9, B2.10 and B2.11, to allow the user to evaluate the reasonableness of the reported FS.

The program implements the algorithm by Zhu et. al., 2002 for performing the GLE iterations (Zhu, D.Y., “A concise algorithm for computing the factor of safety using the Morgenstein-Price method”). The algorithm is expanded to include additional arbitrary loads as well as soil nail and tieback resistance forces.

3. SPENCER METHOD

The Spencer method is analyzed in the same manner as the GLE method within the software program. The only difference is that a single interslice angle is assumed for all the slices (Figure B.2).
C. SOIL NAILING

1. DESCRIPTION OF THE TECHNIQUE

C.1.1. Definition of a soil nailed wall

Constructing a soil nailed wall involves reinforcing the soil as work progresses in the excavated area by the introduction of passive bars, which essentially work in tension. These are usually parallel to one another and slightly inclined downward. These bars can also work partially in bending and by shear. The skin friction between the soil and the nails puts the latter in tension, and transfers acting soil forces deeper within the soil mass.

Using this method, and working from the top downward, a mass of reinforced soil is gradually built up. In order to keep the soil from caving in between the bars, some sort of facing needs to be installed. This facing is generally constructed with some shotcrete reinforced by a welded wire mesh. This facing can be vertical, battered to a wide variety of angles, or made up of a series of benches.

The passive bars are often referred as “nails” and the soil reinforcing technique is known as “soil nailing”.

Once constructed, a soil nailed wall shows a certain similarity with a Reinforced Earth wall. However, the fact that a soil nailed wall is been built downward with the soil being reinforced in situ, while a Reinforced Earth wall is constructed by building an embankment that is then strengthened as the work progresses, constitutes an essential difference between the two.

By its definition, a soil nailed wall is difficult to build under the water table. In these circumstances, special procedures must be adopted, such as pumping operations to lower groundwater levels, drainage, etc.

C.1.2. Construction Stages of a soil nailed wall

A soil nailed wall is constructed in successive phases from the top to the bottom, comprising (see Figure C.1):

1) Excavation, generally limited to 1 or 2 meters deep and possibly limited in length depending on the type of ground being stabilized.

2) Introduction of subhorizontal or inclined nails into the in situ soil.
3) Constructing a facing wall on site (shotcrete over a welded wire mesh or fibrous concrete) or installation of precast elements (or panels) that can be architecturally treated in various ways.

During the excavation phase, the soil must remain stable. This calls for some degree of short-term cohesion in the soil, although it does not need to be highly cohesive. According to CLOUTERRE (1991), about 4 kPa cohesion in Fontainbleau sand was sufficient to assure stability of one meter excavation phases in the first CEBTP experimental wall (of the Project CLOUTERRE). If the soil is slightly cohesive, it is possible to carry out the excavation in slots; phases 2 and 3 are carried out in this order (although these can be reversed, i.e. the shotcrete can be applied before the nails are introduced).

Nails can be installed in two principal ways:

- Either by drilling and then grouting with cement grout or mortar in a predrilled hole,
- Or by either percussive methods or vibro-drilling.

In 1989, soil nailing with grouted nais accounted for just over half the total number of square meters of the soil nailed in France.
Other techniques are currently being developed. These combine vibration driving with injection processes. In the United Kingdom and France, certain techniques have appeared that involve driving nails by a compressed air launcher or a pyrotechnic launcher.

The nails are generally made of steel, although other materials have been used (in particular glassfibers). In the case of reinforced shotcrete, the facing wall is constructed to a calculated thickness that depends, mainly, on the grid layout of the nails, but the actual volume of shotcrete, because of over excavation of the plan cross-section used, is often higher. Unlike other techniques, such as Reinforced Earth, the building of a soil nailed wall may involve a different critical phase with respect to local or overall stability. The latter can be lower during the building phase than when the wall is finally built. Local excavation stability during the earthwork phase depends directly on the height of soil excavated, as was shown in the tests and experiments conducted for the CEBTP No.2 experimental wall of the project CLOUTERRE.

Weepholes must always be provided through the facing so that any water infiltrating the structure can drain away. In areas subject to internal hydraulic flows of water, it is appropriate to install drainage measures such as:

- Subhorizontal drains.
- Drainage details, such as geocomposites installed before the facing wall is constructed.

2. GENERAL METHOD FOR STUDYING THE STABILITY OF A SOIL NAILED STRUCTURE

C.2.1. Possibilities and limitations of the technique

The nailing of in situ soils is a technique largely developed for the construction of short-, medium-, or long-term retaining structures in cuts. This is because:

- It readily adapts to site conditions, in so far as it does not need any preliminary excavations beyond the facing of the structure; also, because of the lightness of the equipment used, it is suitable for sites where access is difficult.

- It is suitable for heterogeneous soils, in so far as the height of the excavations, the method for introducing the inclusions, and their density can be fairly easily adapted to the type and resistance of the soils encountered.

- Because of its speed of construction, and particularly because the general excavation work can be performed almost simultaneously with the various phases of construction of the wall.
The main limitations to the use of the technique are caused by local environmental constraints in the immediate vicinity of the structure itself, by special soil conditions to which the technique is not suited, or because there is a water table present.

C.2.2. Limit state design – Assumptions and data

C.2.2.1. Principles of limit states design

The stability of a soil nailed structure is justified in terms of its ultimate limit state by looking at sufficient potential failure surfaces, that may intersect the nails or not, to determine the most critical potential failure surface (Figure C.2).

![Figure C.2: Different types of potential failure surfaces.](image)

The application of limit equilibrium methods to soil nailed walls consists of comparing the forces or stresses resulting from the external actions, with the maximum resisting forces or stresses that can be mobilized in the soil nailed mass, for a series of potential failure surfaces.
C.2.2.1.1. Basic formula

For limit equilibrium methods of slices (Bishop or perturbation method), the equilibrium analysis can be presented using the following general equations:

\[
\Gamma_{s3} \cdot \tau \left( \Gamma_G G + \Gamma_Q Q + \Gamma_{GW} G_W + \Gamma_A F_A + \Gamma_T F_T + \Gamma_R F_R \right) \leq \tau_{max} \left( \text{soil nailed} \right) / \Gamma_m
\]

Where \( \Gamma_{s3} \): method factor that takes account the approximations inherent in the design method
\( \Gamma_G, \Gamma_Q, \Gamma_{GW}, \Gamma_A, \Gamma_T, \Gamma_R \): should not be used (bad factors according to CLOUTERRE)

\( \Gamma_m \): partial safety factors

In this general equation, the formula: \( \tau \left( \Gamma_G G + \Gamma_Q Q + \Gamma_{GW} G_W + \Gamma_A F_A + \Gamma_T F_T + \Gamma_R F_R \right) \) represents the force or stress on the potential failure surface resulting from the combination (marked +) of the actions shown between the parentheses, and where \( \tau_{max} \left( \text{soil nailed} \right) \) represents the force or stress that can be mobilized in the naired soil along the potential failure surface.

The term \( \tau_{max} \left( \text{soil nailed} \right) \) thus “incorporates” any increase (or reduction) in the shear resistance of the soil along the potential failure surface due to the presence of nails, the effect of which is to increase (or reduce) the normal stresses on the potential failure surface.

The following notations are used for the actions:

- \( G \): permanent loads
- \( Q \): variable loads
- \( G_W \): effects of water
- \( F_A \): accidental loads
- \( F_T \): ground anchors forces
- \( F_R \): nail forces (reinforcement)

The actions are shown in the above formula with their representative value, which is either the characteristic value defined on a statistical basis or a value defined by a code (nominal value).

The characteristic value is defined by the ratio of the most portable value and the dispersion coefficient. One takes for the most portable value its arithmetic average. The dispersion coefficient value is determined to ensure that the characteristic value, maximum or minimum, a minimal probability is not achieved (above or below).

Nominal values are carefully fixed by codes on the assumption of known extreme values or on the basis of other values that might be reasonably considered.
The way in which the representative values of each action are calculated is explained in paragraph C.2.2.2. Each action is ascribed a load factor ($\Gamma_G$, $\Gamma_Q$, $\Gamma_{GW}$, $\Gamma_A$, $\Gamma_T$, $\Gamma_R$).

The resistance of a given material is expressed by its characteristic value that, in principle, shows an acceptable probability of not being reached. With regard to soils, the geotechnical engineer will give the characteristic values to be taken into account and that will be combined with the corresponding partial safety factor $\Gamma_m$.

**C.2.2.1.2. How to account for nails and prestressed ground anchors**

The difference between external forces and resistances is conventional and could be subject to different interpretations depending on the design method used and the point of view adopted regarding prestressed ground anchors and nails. In the form shown above, the forces in both nails and prestressed ground anchors are shown as external forces ($F_R$ and $F_t$) through their tangential components on the potential failure surface, and play a role in the resistance $\tau_{\text{max}}(\text{soil nailed})$ with the effect of their normal components on the potential failure surface.

In any limit equilibrium method, the values of the forces in the nails $F_R$ are calculated for each potential failure surface. This is done by taking account of the nails’ resistances and the soil-nail interaction (one might, for example, use the multicriteria approach [ see paragraph C.2.3.2. of this chapter]). This is why the load factor $\Gamma_R$ of $F_R$ will be written as the inverse form of the partial safety factor of the nails:

$$\Gamma_R = \frac{1}{\Gamma_{m,R}}$$

With the framework of ultimate limit state design, the current rules consider that the forces present in the prestressed ground anchors are external forces and are therefore known; they are independent of the potential failure surface considered.

In the ultimate limit state calculation, it may be appropriate to take as the tensile value of a prestressed ground anchor the smallest value between the guaranteed elastic limit of the reinforcing bar and the anchor limit pull out force. Moreover, steps should always be taken to verify the compatibility between the corresponding total displacement (soil + tie-back) and the deformations experienced by the nail and the soil at failure. Since this design procedure is recognized as complex, the value of the tension will be taken as equal to the lock-off tension, $T_b$, at the same time bearing in mind the soil creep and the steel’s “relaxing”.

The load factor $\Gamma_T$ of the tensile force $F_T$ in the prestressed ground anchor will be taken as being equal to 1, given that the tension at failure will be higher than the loc-off tension.
C.2.2.2. Actions

C.2.2.2.1. Types of actions

The actions to be considered are as follows:

a) **Permanent forces (G)** of the soil’s own weight, either in situ or brought in, and of any structures that form part of the site and/or that affect it, loads caused by buildings located in the structure’s area of influence, and long-term surcharges.

b) **Variable forces (Q)** might include:

   - The effects of rolling loads, vibrations, cyclic loads.
   - Climatic effects (for example, effects of ice on the heads of the ground anchor rods).

c) **The effects of water (GW)** resulting from pore water pressure in the soil nailed mass. In principle, soil nailed structures must, insofar as possible, have drainage facilities in their area of influence. If, in spite of everything, pore water pressure remains, this absolutely must be taken into account.

d) **Accidental forces (FA)** may be caused by:

   - Earthquakes and impacts.
   - Exceptional hydraulic conditions (flooding).

e) **Forces in the nails (F_R)**

f) **Forces in the ground anchors (F_T)**

C.2.2.2.2. Characteristic values of these actions

The minimum or maximum characteristic values given for a specific action will be chosen depending on whether its effect on the structure is to stabilize or destabilize it with respect to the considered potential failure mechanism.

In the absence of any specifications to the contrary being given in the contract documents, the following characteristic values of actions will be used in the above formulae.

1) **Forces due to weight**

For soils and other materials likely to be included in a soil nailed structure, the following characteristic unit weight values will be used:
- Soils

Unit weights are assessed on the basis of representative measurements. In the absence of such measurements, the following nominal values may be used on condition that these clearly result in improved safety of the structure (table C.1).

**Table C.1. Nominal typical Soil density values.**

<table>
<thead>
<tr>
<th>In situ State</th>
<th>Unit Weight (KN/m³) loose</th>
<th>Unit Weight (KN/m³) dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silt</td>
<td>17</td>
<td>20</td>
</tr>
<tr>
<td>Clay</td>
<td>17</td>
<td>19</td>
</tr>
<tr>
<td>Marl</td>
<td>20</td>
<td>22</td>
</tr>
<tr>
<td>Sand</td>
<td>18</td>
<td>20</td>
</tr>
<tr>
<td>Gravel</td>
<td>18</td>
<td>21</td>
</tr>
<tr>
<td>Chalk</td>
<td>17</td>
<td>19</td>
</tr>
<tr>
<td>Weathered rock</td>
<td>20</td>
<td>22</td>
</tr>
</tbody>
</table>

- Reinforced concrete: \( \gamma_{b\min} = \gamma_{b\max} = 25 \text{ KN/m}^3 \)
- Steel: \( \gamma_{\min} = \gamma_{\max} = 78.5 \text{ KN/m}^3 \)

The consistency principle requires that one material is considered to have the same typical unit weight values, whatever its effects, stabilizing or destabilizing, with respect to the considered potential failure surface.

- **Effects of water (G_w)**
Pore water pressures are calculated from the most critical flow net by taking as the unit weight of the water: \( \gamma_{\min} = \gamma_{\max} = 10 \text{ KN/m}^3 \)

- **Forces of prestressed ground anchors (F_T)**
The tensile forces \( F_T \) in the prestressed ground anchor will be calculated on the basis of the procedure explained in paragraph D.2.2.1.2. It is recommended that the following equation be adopted: \( F_{T,\min} = F_{T,\max} = T_b \) (lock-off load)

- **Forces of the nails (F_R)**
The characteristic values of the forces in a nail will be taken to be equal to the limit forces in that nail determined, for example, on the assumption of a multicriteria approach and using characteristic material resistance values (see paragraph C.2.3.2. of this chapter).
2) **Variable forces (Q)**

The characteristic values of variable loads are defined in the contract documents. If this is not the case, take as the characteristic values of the variable load on a platform:

\[ F_{Q,\min} = 0 \text{ kPa} \]
\[ F_{Q,\max} = 10 \text{ kPa} \]

3) **Accidental actions of type (FA)**

- **Seismic loading**

Forces of a seismic origin, specified in the contract documents or by current regulations, are given by a nominal acceleration value \( \alpha_n \) and by a topographic coefficient \( \omega_t \). From these values, one deduces the maximum characteristic values of the seismic coefficients to be applied to the actual weight of all or part of the structure. These coefficients are:

\[ K_H = \pm \frac{\alpha_n}{g} \omega_t \quad \text{(horizontal component)} \]
\[ K_V = +0.5 K_H \quad \text{(vertical component)} \]

The plus sign corresponds respectively to an outward horizontal and a downward vertical.

The minimum characteristic values are \( K_H = 0 \) and \( K_V = 0 \)

**C.2.2.2.3. Combination of actions and calculations**

The combination of actions and calculations to be considered are:

- **Fundamental combinations:**

\[ \Gamma_{s3} \cdot \tau \left( \Gamma'_{s1} G_{\max} + \Gamma_{s1} G_{\min} + G_W + \Gamma_Q Q + \Gamma_T F_T + \frac{F_R}{R_{m,R}} \right) \]

- **Accidental combinations:**

\[ \tau \left( G_{\max} + G_{\min} + G_W + Q + \Gamma_A + \Gamma_T F_T + \frac{F_R}{R_{m,R}} \right) \]

Where

- \( G_{\max} \): permanent forces having a destabilizing effect,
- \( G_{\min} \): permanent forces having a stabilizing effect,
- \( \Gamma'_{s1} \): load factor for the \( G_{\max} \) force
- \( \Gamma_{s1} \): load factor for the \( G_{\min} \) force
For reasons of simplification, only one basic variable force will be considered in all the combination of actions.

The load factors values of the actions are given in Table C.2.

It must be remembered that the forces of the nails FR are reduced by the partial safety factor $\gamma_{m,R}$, which applies to the nail’s failure criterion.

- **Remarks**

1) $\gamma_{s1} = 1.05$ for unfavorable gravitational forces and $\gamma_{s1}’ = 0.95$ for favorable gravitational forces.

   These values differ from those applied to other permanent forces ($\gamma_{s1} = 1.2$ and $\gamma_{s1}’ = 0.9$), and this is justified because gravitational forces, which are dominant in soil nailed structures, are known with a fair degree of accuracy. Uncertainties may arise with the geometries involved (ground dimensions, excavation elevations, etc.). However, given the difficulty in characterizing the self weight of the insitu soil, a $\gamma_{s1} = 1.0$ is adopted in the software.

2) The consistency principle partial safety factors dictates that a single volume of soil be considered with the same characteristic unit weight value, as well as the same partial safety factors $\gamma_{s1}$ or $\gamma_{s1}’$, whatever the considered potential failure surface.

3) A partial safety factor equal to 1 will be used when determining the unit weight of water. The calculated buoyant unit weight value will be equivalent to:

   \[
   \gamma' = \gamma_{s1} \gamma - \gamma_w \quad \text{(overall destabilizing force)}
   \]

   \[
   \gamma' = \gamma_{s1}’ \gamma - \gamma_w \quad \text{(overall stabilizing force)}
   \]

4) $\gamma_{GW} = 1$. This again refers to the consideration that, for simplification purposes and in the case of forces linked to pore water pressures or to flows of water that will have to be taken into account, it will be assumed that safety will have already been accounted in the representative GW values.
C.2.2.3. Resistances

C.2.2.3.1. Failure criteria of materials

The application of limit equilibrium methods to soil nailed structures requires the compatibility of the deformations at failure of the soil and the nails (ductility of the nails, the soil and the soil-nail interface).

<table>
<thead>
<tr>
<th>NATURE OF FORCES</th>
<th>NOTATION</th>
<th>LOAD FACTORS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Fundamental Compination</td>
</tr>
<tr>
<td>1) Permanent forces, type G.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil unit weight</td>
<td>G</td>
<td>$\Gamma_{s1} = 1.05$</td>
</tr>
<tr>
<td>De-stabilizing force</td>
<td></td>
<td>$\Gamma_{s1}' = 0.95$</td>
</tr>
<tr>
<td>Stabilizing force</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other permanent forces</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unfavorable forces</td>
<td></td>
<td>$\Gamma_{s1} = 1.20$</td>
</tr>
<tr>
<td>Favorable forces</td>
<td></td>
<td>$\Gamma_{s1}' = 0.90$</td>
</tr>
<tr>
<td>Water pressures</td>
<td>$G_W$</td>
<td>$\Gamma_{GW} = 1.0$</td>
</tr>
<tr>
<td>Nail force force</td>
<td>$F_R$</td>
<td>$1 / \Gamma_{m,R}$</td>
</tr>
<tr>
<td>Ground anchor</td>
<td>$F_T$</td>
<td>$\Gamma_T = 1.0$</td>
</tr>
<tr>
<td>2) Variable forces, type Q (live loads, climatic forces)</td>
<td>Q</td>
<td>$\Gamma_Q = 1.33$</td>
</tr>
<tr>
<td>3) Accidental forces, type $F_A$</td>
<td>$F_A$</td>
<td></td>
</tr>
<tr>
<td>Method factor</td>
<td></td>
<td>$\Gamma_{s3} = 1.125$</td>
</tr>
</tbody>
</table>

* See Table C.3 – Design Resistances

It will be subsequently assumed in the presentation of design methods that this compatibility exists both for standard nails and the majority of soils. All limit equilibrium methods take into account only the following failure criteria for the materials.
The soil is characterized by a Mohr-Coulomb type criterion where $\phi$ is the friction angle and $c$ is the cohesion. As long-term conditions are always less favorable, one shall, when studying the internal and external stability of a soil nailed wall following completion, take for $\phi$ and $c$ the properties $\phi'$ and $c'$ in drained conditions, whether or not the soil is saturated. For temporary phases, depending on the soil water content, one may use the short-term characteristics $\phi_u$ and $c_u$ of the saturated soil or the characteristics $\phi$ and $c$ of the nonsaturated soil at the in situ water content measured with a triaxial apparatus.

- **Nails**

  The nails will be characterized by the three following resistances:

  $R_n$ – resistance to simple tension,
  $R_c$ – resistance to shear force,
  $M_0$ – moment of plastilication of the nail in pure bending, which will be determined as shown below.

- **Soil-nail interaction**

  With regard to soil-nail interaction, two criteria relating to the two modes of interaction will be examined:

  - The limit skin friction, which will be characterized by the unit skin friction $q_s$.
  - The ultimate bearing pressure $p_u$ under the soil of the nail, which will be taken to be equal to the limit pressuremeter pressure $p_i$. 

C.2.2.3.2. Characteristic values of strength parameters

The characteristic values of strength parameters of the soil and the soil-nail interactions will be taken to be equal to the most representative average values.

- **Soil**

  It will be a geotechnical engineer’s responsibility to define the characteristic values for the shear strength parameters of soils. These must take account of the dispersion, quality, and representativeness of test results.

  With regard to the soil, one shall take the long-term characteristic values of the internal friction angle $\phi'$, and cohesion $c'$, determined based on Tests carried out either in situ or in a laboratory.

- **Nails**

  Where nails include a metal reinforcing bar sealed in grout, the strength of the grout will not normally be taken into account, except where this can be specifically justified with the regulations on reinforced concrete (BAEL 83).

  The characteristic nail strength values ($R_n$, $R_c$ and $M_0$) will be calculated on the basis of the guaranteed elastic limit $\sigma_e$ of the steel where the nails include a metal reinforcing bar.

- **Soil-nail interaction**

  At the project design stage, the characteristic value for the soil nail unit skin friction $q_s$ will be determined based on the charts provided in the appendices of th

  At the construction stage, the value of $q_s$ will be determined from the compulsory pull-out tests. With regard to the resistance of the soil against the nail, the characteristic value of the ultimate lateral pressure $p_u$ of the nail on the soil will be taken to be equal to the limit pressuremeter pressure $p_l$.

C.2.2.3.3. Calculation values of strengths

Strength calculation values to be used for justifying the structure will be determined from the characteristic values by reducing them with a factor $\Gamma_m$, called “partial safety factor”:

$$\text{Calculation value} = \frac{\text{characteristic value}}{\Gamma_m}$$

The $\Gamma_m$ factor values, both in fundamental and accidental combinations, are shown in table C.3.
1) Shear resistance of the soil

For the shear resistance of the soil $\tau_{max} = c + \sigma \tan \varphi$, the partial safety factors $\Gamma_{m,c}$ and $\Gamma_{m,\varphi}$ are applied respectively to $c$ and $\tan \varphi$, which gives the following resistance calculation value:

$$\tau_{max} = c/\Gamma_{m,c} + \sigma \tan \varphi /\Gamma_{m,\varphi}$$

The coefficients $\Gamma_{m}$ proposed for the shear resistance of the soil in particular take account of:

- Any potential differences between the resistance values of the soil in the structure and those determined from the various tests carried out either in a laboratory or in situ.
- Any potential consequences for the structure from an area of soil having a local resistance lower than the characteristic values.

It is appropriate to remember, as in the analysis of slope stability, that the design for a soil nailed structure is extremely sensitive to the values taken to be characteristic of the shear strength of the soil, in particular its cohesion. This justifies the adoption of different partial safety factor values for both the angle of internal friction and the cohesion.

2) Normal soil-nail interaction

a) The partial safety factor $\Gamma_{m'pl}$ values proposed for the limit pressumeter pressure value $p_l$ are:

- $\Gamma_{m'pl} = 1$ for short-term loadings, in particular for excavation phases,
- $\Gamma_{m'pl} = 2$ for permanent loadings, which brings us back to the fact that the ultimate pressure of the soil in contact with the nail is close to the critical creep pressure.

b) The partial safety factor $\Gamma_{m'EM}$ for the pressuremeter module EM, which plays a part in the determination of the subgrade reaction coefficient $k_s$ will be taken to be equal to 1.0 for all combinations, always provided that the value used by the geotechnical engineer for $E_M$ results from a sufficient number of representative tests.

3) Soil-nail unit skin friction: $q_s$

The values of $\Gamma_{m,q_s}$ will depend on the way of determining the characteristic soil-nail unit skin friction either from charts or from in situ pull-out tests.

It will be noted that the values suggested for $\Gamma_{m,q_s}$ are higher than those used for calculating deep foundations. This is due to the extreme sensitivity of this parameter to the conditions of installing the nails.
4) Steels

One shall take $\Gamma_{m,e} = 1.15$ for reinforced concrete bars and other steel with an elastic limit lower than 500 MPa, in accordance with existing regulations.

C.2.2.4. Situations

The whole of the structure must be justified for the situations described below.

- *In course of construction*
  This corresponds to the excavation and earthwork phases and the gradual installation of the reinforcements.

One phase that should be checked is where the earthworks for a section of excavation is completed, although neither the nails nor the facing have been installed.

It will be noted that it may be necessary to look at a set of particular parameters when the structure is in this phase of construction, i. e., for different soil resistances and hydraulic considerations.

- *In service*

This refers to a finished structure

- *In an “accidental” situation*

This situation might correspond to the following situations;

- Earthquake
- Exceptional hydraulic conditions.

One shall look at situations from the point of view of an accidental combination of actions.
### Table C.2

<table>
<thead>
<tr>
<th>MATERIAL PROPERTIES</th>
<th>PARTIAL SAFETY FACTORS ( \Gamma_m ) applied to characteristic values of the materials</th>
<th><em>( \Gamma_m ) standard sensitive</em></th>
<th><em>( \Gamma_m ) standard sensitive</em></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fundamental combinations</td>
<td>Accidental combinations</td>
<td></td>
</tr>
<tr>
<td><strong>1) SOIL</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tangent of the effective friction angle ( \tan \phi^1 )</td>
<td>( \Gamma_m, \phi )</td>
<td>1.20</td>
<td>1.30</td>
</tr>
<tr>
<td>Effective cohesion ( c^1 )</td>
<td>( \Gamma_m, c )</td>
<td>1.50</td>
<td>1.65</td>
</tr>
<tr>
<td>Undrained cohesion ( (\phi_U = 0) ) ( c_U )</td>
<td>( \Gamma_m, c_u )</td>
<td>1.30</td>
<td>1.40</td>
</tr>
<tr>
<td><strong>2) MILD STEEL</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elasticity limit ( \sigma_c )</td>
<td>( \Gamma_m, \sigma_e )</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td><strong>3) SOIL-NAI INTERACTION</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit skin friction (tests) ( q_s )</td>
<td>( \Gamma_m, q_s )</td>
<td>1.40</td>
<td>1.50</td>
</tr>
<tr>
<td>Unit skin friction (charts) ( q_s )</td>
<td>( \Gamma_m, q_s )</td>
<td>1.80</td>
<td>1.90</td>
</tr>
<tr>
<td>Limit pressumeter pressure ( p_l )</td>
<td>( \Gamma_m, p_l )</td>
<td>1.90</td>
<td>2.00</td>
</tr>
<tr>
<td>Pressumeter modulus ( E_m )</td>
<td>( \Gamma_m, E_m )</td>
<td>1.90</td>
<td>1.00</td>
</tr>
</tbody>
</table>
3. CALCULATION METHODS

C.3.1. Stages of calculation

- **First design**
  For the preliminary design of a structure, the most critical potential failure surface should be sought by simultaneously looking at internal, mixed, and external failures.

- **Irritations and optimizations**
  Where a structure is unstable, the project will be modified as many times as is necessary, and its verification performed again. If the structure is stable, the design already verified shall be kept, or optimized by making modifications and reverifying it.

C.3.2. Determination of nail forces at failure (‘multicriteria” rule)

C.3.2.1. Failure criteria and limit equilibrium methods

The forces in the nail at its point of intersection O with the potential failure surface can be represented by a system of forces:

\[
T_n = \text{normal force},
T_c = \text{shear force},
M = \text{bending moment}.
\]

The determination of the forces and moment at which the nails fail, requires the consideration of four failure criteria for the constituents and their interaction with one another:

- Soil-nail friction interaction: \( \tau \leq q_s \)
- Soil-nail lateral pressure interaction: \( p \leq p_u \)
- Soil-nail friction interaction: \( \tau \leq k \)

Where \( k \) is the maximum shear stress of the material from which the nail reinforcement is made.

1) **For methods based on yield design theory**, it is theoretically possible to take into account both the bending moment and the shear forces in the nails. To date, this aspect has not been developed and only the tensile strength of the nails has been studied. Such a calculation would lead to the determination of the maximum resistance of the nails being dependant first on the soil-nail criteria:

Skin friction: \( \tau \leq q_s, \quad \text{ie } |T_n| \leq q_s \pi D L_a \)
lateral pressure: $p \leq p_u$
and, second, by the failure criteria of the nails. The following simplified formula can be used (Anthoine, 1987):

$$\left(\frac{T_n}{R_n}\right)^2 + \left(\frac{T_c}{R_c}\right)^2 + \left|\frac{M}{M_0}\right| - 1 \leq 0$$

Which is slightly conservative when compared with Sobotka’s formulae (1954,1955):

$$\left(\frac{T_n}{R_n}\right)^2 + \left(\frac{T_c}{R_c}\right)^2 + \left|\frac{M}{M_0}\right| \sqrt{1 - \left(\frac{T_c}{R_c}\right)^2} - 1 \leq 0$$

or Neil’s (1961):

$$\left(\frac{T_n}{R_n}\right)^2 + \left(\frac{T_c}{R_c}\right)^2 \left[1 - \left(\frac{T_n}{R_n}\right)^2\right] + \left|\frac{M}{M_0}\right| - 1 \leq 0$$

Which is also considerably easier to use.

With regard to the criterion of lateral pressure ($p \leq p_u$), its formulation in relation to $T_c$ and $M$ requires, by contrast, an assumption to be made about the distribution of the pressure along the nail (uniform distributions, opposed from one side to another of the failure surface (Figure C.3). The validity of this assumption needs to be verified by experimental data.

**Figure C.3:** Schematic distribution of the lateral pressure along the nail.
The maximum contribution that these criteria allow can be calculated in a similar way to that currently proposed for nails that work only in tension. In other words, by verifying the overall equilibrium of the cross section of the structure, which is bordered by a potential failure surface.

At present, these methods are limited in that only the nail’s tensile strength is taken into account. However, where the nails are to be implemented without the introduction of any additional assumptions, this would result in considering the potential failure surfaces as logarithmic spirals (of angle \( \phi \)) where it would be sufficient to check the moment equilibrium at the focus of the spiral. As with classical limit equilibrium methods, one is seeking for the volume of reinforced soil showing the most critical stability conditions.

2) **For classical limit equilibrium methods**, the combination of the materials’ failure criteria and their interactions, as well as the relationships between \( T_n \), \( T_c \) and \( M \), have been studied within the framework of an elastoplastic behavior of the soil-nail system. The relevant multicriteria approach has been in use for several years to design real structures in reinforced soils approach (soil nailing and micro-piles). Four criteria that correspond to four nail failure modes will be considered.

- **The soil-nail skin friction criterion (C1)**

  This criterion, which corresponds to the structure’s failure when the nails are pulled out, is represented for a homogenous soil as:

  \[
  T_n \leq q_s \pi D L_a
  \]

  Where \( q_s \) is the soil-nail unit skin friction.

  \( \pi D \) is the perimeter of the nail where \( D = D_c \) (borehole diameter) for grounded nails, and \( D = D_a \) (equivalent diameter) for driven nails.

  \( L_a \) is the nail grouted length beyond the failure surface, except where there is no facing or connection between the head of the nail and the facing. In this case \( L_a = L^* \), the length \( L^* \) is the shorter of the two lengths between the failure surface and the facing or extremity of the nail in the structure (Figure C.4).
Figure C.4. Determination of pull out length $L_a$.

- a. Nails connected to facing.

$$L_a = L^* = \min\{ L_1, L_2 \}$$

- b. Nails free at their heads.
Soil-nail lateral pressure criterion (C2)

The lateral pressure exerted by the nail on the soils is limited by the ultimate lateral pressure of the soil $p_u$. Failure by bearing pressure of the soil under a nail may be defined either when the $p_u$ is achieved at the single point of maximum shear force (Figure C.5) (most of the conservative assumption), or when the soil is plastification over a maximum length to be defined. In the first (the simplest and most conservative case), an analysis of the nail under combined loading (normal forces, shear force and bending moment) gives us the following criterion:

$$T_c \leq T_{c2,\text{max}}$$

With

$$T_{c2,\text{max}} = \frac{D_c}{2} p_u l_o$$

Where $D_c$ = diameter of the nail (grout and reinforcement)

$l_o$ = transfer length

$p_u$ = ultimate lateral pressure.

In the second case, it will be assumed that the extent of soil plastification under the nail is limited to the value $\pi l_o / 2$, which corresponds to the distance between the two points of maximum moment as determined by the elastic behavior of the nail and soil (Figure C.5)

In both cases, the result is a criterion that focuses on the shear force $T_c$ of the type $T_c \leq T_{\text{max}}$.

Figure C.5. Schematic representation of the soil-nail interaction (elastic behavior).
• Criteria (C3) and (C4)

The two criteria (C3) and (C4) involve the forces and moments \( (T_n, T_c, \text{ and } M) \) created in the nail when it plastifies either by shearing at \( O \) (the point of maximum shear force), or by bending moment at \( A \) and \( A' \) (maximum moment points). To simplify matters, one should assume that the normal force \( T_n \) does not vary in the bending zone around the potential failure surface.

Use Anthoine’s criterion (1987) to represent the actual resistance of the nail, since this is both simple and slightly conservative:

\[
\left( \frac{T_n}{R_n} \right)^2 + \left( \frac{T_c}{R_c} \right)^2 + \left| \frac{M}{M_0} \right| - 1 \leq 0
\]

➢ Criterion (C3)

Nail plastification by shearing occurs at the point of maximum shear force \( O \). This corresponds (for reasons of symmetry) with the point of nail / failure surface intersection, provided the nail lengths between one side and another on the potential failure surface are at least longer than \( 3l_0 \).

At the \( O \) point, the bending moment is zero \( (M = 0) \) and the failure criterion, based on the general failure criterion of the nail, can be written as:

\[
\left( \frac{T_n}{R_n} \right)^2 + \left( \frac{T_c}{R_c} \right)^2 \leq 1
\]

One usually takes \( R_c = R_n / 2 \).

In the \((T_n, T_c)\) plane, (C3) is represented by an ellipse.

➢ Criterion (C4)

Using a simplified assumption, nail plastification by bending moment occurs at the points of maximum moment \( A \) and \( A' \) located on both sides of the potential failure surface at a distance equal to \( l_p = \pi l_0 / 4 \) and calculated with an elastic behavior of both soil and nails.

Plastification at those points (see Figure C.2) where the shear force is zero \( (T_c = 0) \), corresponds to the friction:

\[
M \leq M_{max}
\]

With \( M_{max} = M_0 \left[ 1 - \left( \frac{T_n}{R_n} \right)^2 \right] \) determined by the nail failure criterion.

Based on this value, the following formula gives the shear force at point \( O \):
\[ T_{co} = a \frac{M_0}{l_0} \left[ 1 - \left( \frac{T_n}{R_n} \right)^2 \right] \]

Where \( a \) is a constant and is equal to 3.12.

In practise, plastification at the two maximum moment points A and A' in the initial phase does not imply the failure of the system. Plastification remains localized with two plastic hinges in the nail. These hinges, which initially occur at A and A', move as the nail continues to deform. The value of \( l_p \) is initially equal to \( \pi l_0 / 4 \) but then varies in order to meet equilibrium equations and the nail’s failure criterion. The calculation of \( l_p \) in the elastoplastic case is complicated, and certain experiments tend to show that \( l_p \) varies even inside the same structure. In the absence of any more detail information, a simple assumption involves taking \( l_p \) as constant and equal to \( \pi l_0 / 4 \). At point O, after the two pastic hinges at A and A' have developed, the plastification of the soil under the nail yields the following criterion:

\[ T_c \leq T_{c4,\text{max}} \]

With

\[ T_{c4,\text{max}} = b \frac{M_0}{l_0} \left[ 1 - \left( \frac{T_n}{R_n} \right)^2 \right] + c D_c l_o p_a \]

Where \( b \) and \( c \) are two constants and equal respectively to 1.62 and 0.24.

The criterion respects the equilibrium equations but not the nail’s failure criterion. However, by combining (C3) and (C4), the latter criterion is met, and this is the (conservative measure to be adopted when determining the multicriteria approach.
D. SUPPORT FORCES

Bracing support forces can contribute to increasing the overall global stability. For a support to be beneficial for a particular failure surface, the overall external force it applies on the soil mass must be against the movement direction. If a support is all included within a failure surface then no external loading is applied. Depending on the selected analysis method, and user preferences, different approaches are available for modeling the effects of support forces on slope stability.

First of all, one has to select the design force magnitude to be included in the analysis. Support forces can be ignored or included at the service reaction level, design capacity level, or at ultimate capacity. Table D.1 describes these options in further detail:

<table>
<thead>
<tr>
<th>Option</th>
<th>Description</th>
<th>Important Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ignore support forces</td>
<td>All support forces are ignored for slope stability analysis purposes.</td>
<td>Very conservative approach.</td>
</tr>
<tr>
<td>Include service reactions</td>
<td>All support forces are included at the service reaction level. This is the reaction as calculated from the lateral earth pressure wall analysis (limit-equilibrium or non-linear).</td>
<td>Sometimes it is prudent to include the service reactions for all supports due to displacement compatibility issues between soil mass movements and supports. For example, it is well known, that inclined ground anchors may experience load relaxation with increasing vertical displacement.</td>
</tr>
<tr>
<td>Include design capacities</td>
<td>Support forces for slope stability are included at the design level (for ultimate codes such as EC7 this could be the same as the ultimate capacity). For allowable codes the modeled forces include safety factors from the wall analysis.</td>
<td>The minimum capacity between the design geotechnical and the design structural capacity is always considered. For ground anchors, the analysis considers if the fixed part is intersected by the examined failure surface and the support force is adjusted accordingly.</td>
</tr>
<tr>
<td>Include ultimate capacities</td>
<td>Support forces are included at the ultimate capacity level.</td>
<td></td>
</tr>
</tbody>
</table>
D.1 SUPPORT FORCES IN BISHOP METHOD

In the Bishop method, only circular failure surfaces should be examined. One of the major limitations of the Bishop method is that only moment equilibrium is considered. To simplify the analysis and verification, support effects in the Bishop method are calculated as an additional moment resistance that is applied on the overall moment safety factor equation. The moment resistance is calculated for each support about the center of rotation. If a support is encompassed by the failure circle then this support is ignored for slope stability purposes.

![Figure D.1: Sample circular failure with Bishop method and a ground anchor.](image)

\[ M_{res} = F \times (Zc - Z_{support}) \times \cos(\alpha) \]

If a tieback fixed body intersects a failure surface, then that slice experiences increased normal base reaction due to the tieback. This additional shear resistance is considered by the program by:

\[ N = F \times \cos(\beta) \]
\[ \tau = N \times \tan(\varphi') \]

Where:
- \( F \) = Effective ground anchor or support force to be considered
- \( N \) = Additional normal reaction
- \( \tau \) = Additional base shear resistance

The additional moment resistance is considered by multiplying the base shear resistance by the failure surface radius.
D.2 SUPPORT FORCES IN GLE AND SPENCER METHODS

In the General Limit Equilibrium or the Spencer method, the support forces are considered in the overall vertical and horizontal directions for both moment and horizontal equilibrium. Since supports are always installed first on a vertical face of the wall slice, the general equilibrium equation with the moments (B2.7) is expanded to include the exact point where each support force is applied. The total vertical and horizontal support force is included (Figure D.2).

![Graph showing slope stability slice 16 force polygon](image)

**Figure D.2:** Sample results for a slice at a wall with a ground anchor of Figure D.1.

Since the total force is first applied on the wall slice, in the intersecting slice we only have to consider the force that is within the slice (negative Fx in Figure D.3). With this approach the program will redistribute the support forces for the wall slice from the iterative equilibrium routine.
Figure D.3: Sample results for the slice where the ground anchor intersects the failure surface.
E. REFERENCES

