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Review

Flexible systems anchored to the ground for slope stabilisation: Critical review of existing design methods.

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Abstract

The aim of this article is to review and analyse the different hypotheses assumed in the calculation methods for flexible systems used in slope stabilisation.

These systems are formed by a membrane (cable net or high-resistance wire mesh) and anchored bolts. Several manufacturers and independent researchers assume that the membrane can stabilise the slope by exerting a normal pressure, which leads to an increase in the shear resistance of the ground: This system behaviour is denominated ‘active’. The two main conditions that flexible systems have to fulfil to be considered active (to avoid detachment or sliding from being produced) are that the membrane should be pre-tensioned when installed and that the slope must have a convex curvature. None of the manufacturers-installers verify the membrane’s pre-tension force and moreover, in many cases, the membrane does not have a convex curve, but may be planar or even have a concave one. Additionally, the force applied on bolts to tighten them does not usually exceed 50 kN. Thus, these systems do not work actively, but passively; which means they are able to retain a mass of soil or a rock piece when the sliding has already occurred, but they are unable to prevent it.

Therefore, current design methods used by manufacturers and researchers can be incorrect, leading to extra installation costs in the flexible system in some cases or even an unsafe solution in others.

Keywords

Slope stabilisation, cable nets, wire meshes, passive systems, active systems.

1. INTRODUCTION

Flexible systems anchored to the ground constitute a technique for slope surface stabilisation. These systems are formed by membranes, made of cable nets or wire meshes, and bolts anchored to the ground. This technique has spread extensively due to its low visual impact and its minimal influence on traffic during installation.

Flexible systems may be classified as either active or passive. Active systems attempt to prevent rock detachment or soil sliding, as they apply a pressure on the ground through an initial pre-tension of the flexible membrane that covers the unstable zone. In contrast, passive systems employ very low rigid membranes which are not pre-tensioned during installation; so, they are unable to exert any initial pressure on the ground. Among the active flexible systems on the market, we can find cable nets, manufactured by different companies with very similar characteristics, and also single-torsion high-resistance wire meshes.

Passive systems were first used in the 50s (Peckover, 1976), while active ones were introduced in the 80s (Justo, 2009). Although the use of active flexible high-resistance systems has become generalised throughout the world, there is no official technical document to guide the design and calculation of these systems (Bertolo, 2009), except for a brief reference shown in a soil nailing guide published in UK by the CIRIA (Phear et al. 2005). As a result, the manufacturers have proposed many different design methods. Moreover, there are few scientific references tackling the topic of design methodology, except for those of the manufacturers of cable nets and high-resistance wire meshes themselves.

Only two field monitoring campaigns were found in the bibliography, one in the USA (Muhunthan, 2005) and another in Italy (Bertolo, 2009). In the first case, various emplacements with passive systems (no initial pre-tension was applied) were monitored. Strains on reinforced vertical cables located in the upper part of the slope were measured in order to register overloads caused by snow or debris accumulation. In the second case, a force was applied to the membrane, using a hydraulic jack placed on in the rock slope, in order to measure the resistance of the whole system. Load cells were installed in bolts and reinforcing cables to register force at the moment when maximum load was applied. Neither of these methods measured initial pre-tension force on the membrane or in the bolts, so there are no references on the pre-tension force applied in flexible systems.

In this context, it is considered highly important to analyse the calculation hypotheses that existing models are based on, and propose a new design approach that better describes the real interaction between membrane (unstable soil/rock) and stable slope. Therefore, this paper provides a first step in a more extensive project (now under development) whose final aim is to develop a detailed design method for flexible systems anchored to the ground.

2. DESCRIPTION OF THE SYSTEM

As was mentioned above, there are, in general, two types of active flexible systems, cable nets and high-resistance wire meshes. The former are more frequently available, being a common type for most manufacturers. The latter system, is made up of a single-torsion mesh whose wire is thicker than conventional wire meshes.

2.1 Cable nets

Cable nets anchored to the ground (see Figure 1) include three main elements:

– Cable nets: manufactured with braided 8 to 10 mm galvanised steel cable that forms a weave of grids from 200 to 300 mm. The cables are fixed at the intersection points of the net weave by staples. Cable nets are usually provided by manufacturers in square or rectangular panels of different dimensions, with sides from 2 up to 6 m.
– Reinforcement and perimeter cables: employed to fit the net to the ground and make the system rigid through connection to the central bolts and the anchors of the perimeter cable. The diameter size depends on the manufacturer, but varies from 8 to 20 mm. Reinforcement cables are horizontally and vertically distributed, forming a square or rectangular pattern of 2 to 6 m, knitting the cable net panels together. At the intersection points, the horizontal and vertical cables and the membrane are fixed to the ground by a spike plate and a nut screwed in a bolt. The perimeter cables enclose the outer area of the zone to be stabilised.
– Bolts: they are placed at the crossing points of the reinforcement cables.
– Cable anchors: they are used at the edge of the zone to be stabilised to brace and tense the perimeter cables.
– Spike plate: to attach the intersection of the net cables and reinforcement cables to the ground by a screw thread in the bolt, which is placed above the plate.

Once the triple-torsion mesh is set in place, the net is installed. During the installation process, the cable net panels are laid from the top of the slope to the bottom. The panels are fixed to each other either by clamps or by sewing cables, depending on the manufacturer's installation manual. At the corners of the panels, some perforations are made where the intermediate bolts will be placed. A small depression is made around the perforation, so that the reinforcement cables have a slightly convex shape. Additionally, pre-tensioned reinforcement cables are placed vertically and/or horizontally before tightening the intermediate bolts. When using sewing cables between panels, they also generally work as reinforcement cables. The next step is to tense the perimeter reinforcement cables outwards, which helps to pre-tension the net. This process of tensioning is performed both for horizontal and vertical reinforcement cables. Finally, the internal bolts are tightened, attaching the net to the ground in the depression around the bolt, contributing to an additional membrane pre-tension.

2.2 High-resistance wire meshes.

High-resistance wire meshes anchored to the ground (see Figure 2) are composed of the following elements:

– Wire mesh: single-torsion mesh, manufactured with 3-4 mm thick wire. The rhombus size is 143 mm long x 83 mm wide. They are manufactured in rolls, instead of panels.
– Perimeter cables: the perimeter cables enclose the outer part of the zone to be stabilised, although they are not always used.
– Reinforcement cables: their use is optional. When installed, they are generally placed in horizontal lines.
– Bolts: are arranged in lines and columns with a constant separation, but patterns of square panels are not desired. They are used both for the internal zones of the mesh and the outer perimeter.
– Cable anchors: used on rare occasions on the perimeter.
– Spike plates: they fix the mesh to the ground through a screw thread in the bolt. According to a certain manufacturer, the tightening force may reach 50 kN (see Figure 3) on the ground.
– Clips: they are used to join rolls of wire mesh and to give continuity to the membrane.

The system installation process is very similar to the cable meshes, except that reinforcement cables are not always employed, and when used, they are only placed in horizontal arrangements. Another difference is the attachment between rolls: instead of vertical reinforcement cables, clips are used to attach mesh rolls.

3. CURRENT METHODOLOGY OF DESIGN

In this section, eight different design models are described, from three manufacturers and two independent researchers. The manufacturers state in their technical brochures that these systems are considered as *'active'*, preventing soil sliding or rock detachment. In relation to the researchers, their main hypothesis for analysis of slope stability is that the membrane and bolts exert a uniform pressure able to stabilise the slope, which is equivalent to conceiving the flexible system as 'active'.

3.1. Infinite slope, model A (for soils)

This model was proposed by the Spanish researcher Almudena da Costa (2004;2010) in the University of Cantabria. It determines the pressure necessary to exert on a slope surface to stabilise it through an active membrane. It is based on the failure mechanism of an infinite slope, whose solution is available in general soil mechanics textbooks (e.g. Lambe, 1969). It starts with the assumption that the slope is high enough to consider it infinite, so that the interaction forces of the upper and lower slice are equal, and therefore not considered. Assuming a limit equilibrium analysis and applying Coulomb's yield criterion in the failure surface ($\tau = c' + \sigma' \tan\phi$), the stability of any slice can be considered.

The action of the membrane and the bolts can be included in the typical infinite slope model by adding a normal pressure *p* and a shear pressure *t*, which are both evenly distributed along the slope surface, and expressed by slope width unit (see Figure 4) . The value of *t* can be expressed as $t = p \cdot \tan\delta$, where δ is the friction angle between soil surface and membrane. The total force that the bolt can bear will be $F_{bolt} = pl\cos\delta + tlsen\delta$, where *l* is the vertical separation between bolts.

The value of *p* (1) is obtained by solving the equations of equilibrium of forces in two directions in a slice of the slope (see Figure 4). The ground parameters are defined by the density γ , internal friction angle ϕ , cohesion *c* and safety factor *FoS*. Additionally, geometric parameters must be defined, such as unstable layer depth (*h*), slope angle (α) and streamline angle (λ):

$$p = \frac{\gamma \cdot h \cdot \left(sen\beta - \cos\beta \cdot \frac{\tan\phi}{FoS} \right) + \gamma_w \cdot \frac{h \cdot \cos\lambda}{\cos(\beta - \lambda)} \cdot \frac{\tan\phi}{FoS} - \frac{c}{FoS}}{\frac{\tan\phi}{FoS} + \tan\delta} \tag{1}$$

The values of *p* and *t* will then be used to design both bolts and membrane. A table is defined with the theoretical pressure *p* for various input values. In this way, knowing the values of *p* and *t*, a flexible system solution is chosen that stabilises the slope.

Knowing the values of *p* and *t* necessary to stabilise the ground, as well as the nominal resistance of the meshes obtained through laboratory tests and/or numerical simulations, it is possible to choose a flexible system solution (specific combination of membrane + bolts) that stabilises the slope.

3.2. Infinite slope, model B (for soils)

This model is proposed by a manufacturer for the design of the bolts of the flexible system. It is also based on limit equilibrium analysis in an infinite slope. The difference compared to the previous one is that water is not included. In addition, a stabilising shear pressure *S* is added (see Figure 5), which represents bolt shear resistance in order to maintain the equilibrium of the unstable layer. The manufacturer uses this model only to verify the bolt integrity, under both shear and tensile forces, but not to verify the membrane integrity (Guasti, 2003; Flum, 2004). The force *V* (or total force in the bolt direction) represents the pre-tension in the bolts, which are anchored at a certain angle Ψ with respect to the horizontal. In the most general cases, bolts will be tightened by a conventional or dynamometric wrench, reaching about 50 kN (Geobrugg Ibérica 2008). The rest of the parameters are graphically described in Figure 5. Note that *T, N,* ϕ and *c* are related to total pressures and are not effective, because water is not considered. Two force equilibrium equations are established in the slice in addition to the Coulomb yield criterion equation ($T = N\tan\phi + cA$) in order to obtain the three unknowns, *N, T* and *S.* The parameter *FoS* represents a safety factor applied to the maximum shear force on sliding surfaces (*T*). The value of *S* -see (2)-, is used to check the bolt integrity under shear stresses. Bolt integrity under tensile force *V* is verified as well.

$$S = G\sin\alpha - V\cos(\alpha+\psi) - \frac{[G\cos\alpha + V\sin(\alpha+\psi)]\tan\phi + cA}{FoS} \quad (2)$$

Additionally, the manufacturer verifies the membrane stability with two models of local failure, defined in sections 3.4 and 3.6.

3.3. Slope discretised into several wedges (for soils)

A failure mechanism in soil slopes is proposed by Almudena da Costa based on the concept of a planar fracture parallel to the slope. However, decomposition into unstable wedges is applied so that the effect of the slope height is taken into account (see Figure 6). Thus, this is a less conservative alternative to the hypothesis of infinite slope failure mechanism (3.1), which is especially suitable for slopes with a limited height in relation to the thickness of the unstable layer (Da Costa, 2004; Da Costa, 2010). In this model, as well as in the case of infinite slope, the main hypothesis is that the membrane is able to exert a pressure *p* on the ground which avoids the sliding from taking place. In the same way as previous models, limit equilibrium analysis is considered and Coulomb's yield criterion is applied in the sliding surfaces.

In this model, the unstable layer of ground parallel to the slope with thickness *d* is divided into a series of wedges of size *s* (determined by anchor distance), which define sliding planes at an angle *I* with respect to the slope surface. Both wedge dimensions, *d* and *s*, must be defined at the beginning of the calculations. The solution method consists in establishing the force equilibrium from the crest to the toe of the slope, between an upper block (which would accumulate the results previously obtained in equilibrium equations) and its neighbouring lower wedge (Figure 7). In the first step calculation, Block A is formed only by wedge 1, and Block B by wedge 2. In an i-step calculation, Block A is formed by 1, 2,…,i wedges and Block B by wedge i+1. For i-step calculation, 4 equations are established, 2 equations per block, considering equilibrium of forces in two normal directions (slide surface and its perpendicular), and 4 unknowns have to be worked out: N'_1, N'_2 , N'_3 and p_i . The * super index means that the parameter is divided by the safety factor. Water presence is considered, hence normal and shear ground forces are expressed in effective pressures, U_1, U_2 and U_3 being water pressure forces. Parameter k is defined as $k = \left(\sin\lambda + \cos\lambda \cdot \tan\phi'^{*}\right) / \left(\cos\lambda - \sin\lambda \cdot \tan\phi'^{*}\right)$. The rest of parameters are graphically defined in Figure 7.

The pressure necessary to stabilise Block B, p_i, is defined in (6), assuming that reactions N'_1, N'_2 and N'_3 are positive. If a negative value of any reaction (N'_k<0) is obtained, the force equation system should be recalculated assuming N'_k =0, and leaving the corresponding safety factor free for that i-step, in order to solve a compatible equation system.

$$N'_1 = W_1\cos\beta + s\sum_{j=1}^{j=i-1} p_j - c'^{*}d - N'_2\tan\phi'^{*} - u_1 \tag{3}$$

$$N'_2 = \frac{c'^{*}\left(d\tan\phi'^{*} - (i-1)s - \dfrac{d}{\tan\beta}\right) + W_1\left(\sin\beta - \cos\beta\tan\phi'^{*}\right) - s\left(\tan\phi'^{*} + \tan\delta\right)\cdot\sum_{j=1}^{j=i-1} p_j + u_1\cdot\tan\phi'^{*} - u_2}{1-\left(\tan\phi'^{*}\right)^2} \tag{4}$$

$$N'_3 = \frac{c'^{*}\left(2\cdot d\cdot\tan\phi'^{*} - s\right) + N'_2\left(1+\tan\delta\tan\phi'^{*}\right) + W_2\left(\sin\beta + \tan\delta\cos\beta\right) + u_2 - u_3\left(\sin\delta + \tan\delta\cos\delta\right)}{\left(\sin\lambda + \cos\lambda\tan\phi'^{*}\right)\left(1+\dfrac{1}{k}\tan\delta\right)} \tag{5}$$

$$p_i = \frac{N'_2\left(1-k\tan\phi'^{*}\right) + W_2\left(\sin\beta - k\cos\beta\right) - c'^{*}\left(s+2kd\right) + u_3\left(k\cos\lambda - sen\lambda\right) + u_2}{s\left(k+\tan\delta\right)} \tag{6}$$

Note that p_i increases for every step calculation, therefore p_i maximum is at the toe of slope, in the lowest wedge. In practice, the flexible membrane should be designed considering this value.

3.4. Slope discretised in block and wedge (for soils)

This model is proposed by a manufacturer for selecting an adequate product (IberoTalud and Universidad de Cantabria 2005). This failure model, applicable in soils or highly friable rocks, considers an unstable layer parallel to the slope, except at the slope toe, where the fracture is wedge shaped, so that the mechanism is kinematically possible (see Figure 8). Coulomb's yield criterion is applied in the limit equilibrium analysis; thus, it is necessary to know the soil parameters (γ, ϕ, c), unstable layer depth (*h)*, slope height (*H)*, and sliding angle of the lower wedge (α). Normal and shear pressures (*p* and *t*) represent the membrane's contribution to stabilising the ground, but in this case, *t* is also an unknown. Applying Coulomb's yield criterion and taking into account the presence of water, shear interactions between blocks, T_1, T_{12}, T_2, are substituted by $T_i = N_i' \tan\phi / FoS + cA_i / FoS$, where A_i is each sliding surface area and FoS is the safety factor for soil strength parameters. Four equilibrium equations are considered, two in each block, but 5 unknowns: *p, t,* N_1, N_2, N_3, α have to be worked out. In order to obtain the value of *p*, expression (7), which depends on known parameters k_i and α, is maximised, thus providing the fifth equation (8). The p_{max} value obtained is then used to select the specific flexible system (membrane-bolts).

$$p = \frac{\left[W\ A - \frac{c}{FoS \sin\beta}(H + hB) + U_1 \frac{\tan\phi}{FoS} + U_2 C\right] E - \left[W_2 + \frac{ch}{FoS} D - U_{12} \frac{\tan\phi}{FoS} - U_2 G\right] K}{\left(\frac{\tan\phi}{FoS} + \tan\delta\right) \frac{H}{\sin\beta} E + (B - A\tan\delta) \frac{h\cos\alpha}{\cos\beta \sin(\beta - \alpha)} K} = p(k_i, \alpha) \quad (7)$$

$$\frac{dp(k_i, \alpha)}{d\alpha} = 0 \rightarrow p = p_{\max} \quad (8)$$

Where: *W* is the total weight of the unstable soil, W_2 is the lower wedge weight $A = \sin\beta - \cos\beta \tan\phi / F$, $B = \cos\beta + \sin\beta \tan\phi / F$, $C = \cos(\beta - \alpha)\tan\phi / F - \sin(\beta - \alpha)$, $D = L / \sin(\beta - \alpha) + 1/\cos\beta$, $E = 2\sin\alpha \tan\phi / F + \cos\alpha\left(1 - \tan^2\phi / F\right)$, $G = \cos\alpha + sen\alpha \tan\phi / F$, $K = \left(1 + \tan^2\phi / F^2\right) \cdot sen(\beta - \alpha)$, $L = \cos\alpha \tan\phi / F - sen\alpha$.

For high slopes, the solution obtained, p_{max}, is approximately equal to the one considering infinite slope in model A (see 3.1).

When the flexible system is to be installed in rock mass instabilities, the company does not have specific software for the calculations. In this case, the solution is based on the project geological-geotechnical annex; where an average pressure over the surface slope is calculated in order to prevent wedge sliding.

When the total pressure to be applied to the ground is determined, the manufacturer relies on some tables where the maximal resistance of different arrangements of the anchored net is listed, in relation to the grid aperture (200, 250 or 300 mm) and separation between bolts (2, 3 or 4 m). This table is designed using finite element software, where a net panel with a fixed outline is simulated, to which an evenly distributed load was applied (Castro, 2008). The simulation results have been verified by laboratory tests for certain net arrangements (2x2m, grid aperture 200, 250, 300 mm) performed in the Structural Engineering Laboratory at the University of Cantabria, Spain (Castro, 2009).

The selected anchored net must have a maximal resistance that matches with the pressure to be exerted on the slope to avoid sliding of the soil mass.

3.5. Infinite slope, model C (for rocks)

This model was proposed by a manufacturer to design its ground-anchored cable nets, which are considered in its technical brochure as active flexible systems (Officine Maccaferri S.p.A., 2008). The information shown in this paper comes from the manual of the company's (Officine Maccaferri S.p.A., 2006) freely distributed software for facilitating the design of the specific flexible system solutions (membrane+bolts). Its field of application is more focused on instabilities in rock slopes at the moment the failure takes place (limit equilibrium analysis).

The main hypothesis stated by the manufacturer is that there is a layer parallel to the slope with a specific thickness, as represented in Figure 9, where unstable wedges may emerge (Officine Maccaferri S.p.A., 2008). In the software two failure mechanisms are used: in the first one, the slope is considered as infinite with an unstable layer of thickness *s*, and in the second it is considered that local wedges could slide through a specific joint angle *a*. The first failure model, which is described in this section in more detail, is used to calculate the safety factor in bolts, considering that these are the only elements that contribute to the slope's overall stability. The second failure mechanism, which considers a wedge fracture (see paragraph 3.8), is used to calculate the safety factor in the membrane due to normal and shear forces.

To calculate the safety factor of the slope's global stability, the main hypothesis stated is that bolts will be able to stabilise the friable layer by exerting a pressure normal to the ground, thus increasing the friction between the unstable layer and the ground below it. In addition, bolts are assumed to act passively, which means that they can exert pressure when a certain deformation on them has already occurred. That tensile deformation in bolts is a consequence of a specific dilation on joint rock (increase of average joint spacing when sliding is taking place).

Limit equilibrium analysis is applied in an infinite slope of angle b regarding maximum shear stress in the sliding plane using Coulomb's yield criterion, instead of Barton and Choubey's (1977) expression, $\tau = \sigma \tan\left[JRC \log_{10}\left(JCS/\sigma\right) + \phi_B\right]$. Cohesion is not considered in Coulomb's expression, so maximum shear pressure is expressed as $\tau = \sigma \tan\phi$ and a constant frictional angle of 45º is assumed. Seismic acceleration is also considered by assuming a horizontal force acting on each slide of a value *Wc,* where *c* is a seismic coefficient. Water influence is not taken into account.

The manufacturer applies various simplifications when calculating the safety factor *FoS* for the overall slope stability. Firstly, an infinite slope without bolts and seismic acceleration is considered in order to calculate the stabilising forces assuming that the unstable layer is in equilibrium (see Figure 10). Thus, relation (9) is established. Then, the safety factor *FoS* -see (9)- is calculated considering the bolt stabilisation and seismic force contributions, *R* and *Wc* respectively. A partial safety factor γ_{dw} is added for the driving force component of weight and seismic force. The bolt stabilisation force *R* is defined by expression (12). Force *R* is derived by considering an additional contribution of shear force due to an increase in pressure normal to the joint surface. This increase in normal pressure due to bolt elongation is related to joint dilation angle (*JR*) and the angle between the joint normal and the bolt θ. Joint dilation angle *JR* is calculated with expression (13), where *JRC* is the joint roughness coefficient; *JCS* is the joint compressive strength and σ is the normal stress. The dilation angle (*JR* or d_n) is slightly smaller than the lower limit proposed by Barton and Choubey (1977), where $d_n = 0{,}5 JRC \log_{10}(JCS/\sigma)$.

$$Stab.\ forces = Driving\ forces \rightarrow Stab.\ force = Wsen\beta \qquad (9)$$

$$FoS = \frac{Stab.\ forces}{Driving\ \ forces} \approx \frac{Wsen\beta - cWsen\beta \tan\phi + R}{\gamma_{dw} W(sen\beta + c\cos\beta)} \qquad (10)$$

$$\tan\phi \approx 1 \rightarrow FoS = \frac{Wsen\beta(1-c)+R}{\gamma_{dw} W\left(sen\beta + c\cdot\cos\beta\right)} \qquad (11)$$

$$R \approx \frac{16 + 1/\tan^2(\theta + JR)}{4 + 1/\tan^2(\theta + JR)} \qquad (12)$$

$$JR = \frac{1}{3} JRC \log\left(\frac{JCS}{\sigma}\right) \qquad (13)$$

The manufacturer applies the procedure proposed by Panet (1987) to calculate the shear resistance contribution from bolts, *R*. The expression proposed -see (12)-, is based on estimating the maximum principal work on the bolt due to tensile and shear forces. Both actions on the bolt are provoked by joint dilatancy movements. According to Panet, it is assumed that maximum allowable yield tensile stress on the bolt is mobilised.

3.6. Block and wedge limited between two rows of bolts, model A (for soils)

This model, also proposed by a company, verifies the integrity of the membrane (Guasti 2003; Flum et al. 2004). The integrity of the system's bolts was verified by the method explained in section 3.2. It is based on the hypothesis that there is a surface layer in the slope likely to show instabilities, where wedges of ground limited by rows of bolts may emerge. Coulomb's yield criterion is applied in a limit equilibrium analysis.

A local instability mechanism is assumed formed by a lower wedge (Body 2) and an upper block (Body 1) delimited by two rows of bolts (see Figure 11). The thickness of the unstable block, *t*, is assumed to be a known value. The manufacturer assumes a failure mechanism where force *P* represents the force that the membrane exerts on the ground, but acting only on Body 2. It is also assumed that *P* is applied at an angle Ψ with respect to the horizontal, which is equal to the bolt anchoring angle. Force Z is a shear stabilising force on the surface, which represents a pre-tension force on the membrane, also applied only to Body 2. It is assumed to be of a known value. The ground above Body 1 is assumed to be stabilised by the membrane and the bolts. In addition, it is assumed that there is no interaction between Body 1 and the ground above it. Parameter β defines the inclination of the sliding plane of the unstable wedge. The model does not consider the possible presence of water. Applying Coulomb's yield criterion, ground shear interactions T_i can be substituted by $T_i = N_i' \tan\phi + cA_i$, where A_i is each sliding surface area. Additionally, instead of assuming a case of planar deformation (static analysis in 2D, with infinite width), it is considered that there is a specific width of wedge of ground likely to slide, which is not confined by the influence of the pressure of the spike plates. Therefore, when calculating the weights G_1 and G_2, a width a_d is considered, assuming the existence of a radius of influence of the spike plates, as is shown in Figure 12. The parameter F_M represents a safety factor applied to the maximum shear force on a sliding surface (*T*). The rest of geometric parameters are graphically described in Figure 11.

Four equilibrium equations are established, two in each block, where 5 unknowns have to be worked out: N_1, N_2, x, P, β. In order to obtain the value of p, expression (14), which depends on the known parameters k_i and β ,is maximised, providing the additional equation. The value obtained p_{max} is then used to select the specific flexible system (membrane-bolts).

$$P = \frac{G_2\left(\sin\beta - \cos\beta \frac{\tan\phi}{F_m}\right) + \left[G_1\left(\sin\alpha - \cos\alpha\frac{\tan\phi}{F_m}\right) - \frac{c}{F_m}A_1 - Z\right]\left[\cos(\alpha-\beta) - \sin(\alpha-\beta)\frac{\tan\phi}{F_m}\right] - \frac{c}{F_m}A_2}{\cos(\beta+\psi) + \sin(\beta+\psi)\frac{\tan\phi}{F_m}} \tag{14}$$

$$\frac{dP(k_i,\alpha)}{d\beta} = 0 \rightarrow p = p_{\max} \tag{15}$$

3.7. Block and wedge limited between two rows of bolts, model B (for soils)

This model was proposed by Daniel Castro (Castro, 2000), a researcher at the University of Cantabria, in his PhD thesis. The field of application is limited to soil slopes or highly meteorised rock, hence Coulomb's yield criterion is applied in a limit equilibrium analysis.

The failure model considers an upper block and a lower wedge, both of equal length, l, located between two rows of bolts. The model assumes that the ground above the upper block is stable. This model is quite similar to the one described in section 3.6. One of the differences is that block and wedge have equal length. With this additional assumption, there is no need to know a priori the thickness H of wedge and block. An additional hypothesis made is that the surface in between wedge and block is parallel to the bolt direction. However, these two hypotheses are not based on any practical or theoretical argument. In addition, the stabilising shear force Z is not considered in this model.

The membrane is assumed to be able to exert a uniform pressure on the ground, so that it prevents the sliding of the upper block and the lower wedge. That pressure, concentrated over the centre of gravity of the upper block, is referred to as the total force *Q*. In the model, it is also assumed that the total force *Q* exerted by the membrane on the ground is equal to the force that the bolts apply to the ground. Angle θ represents the anchoring angle of bolts. *G* is the weight of the lower wedge expressed in weight per unit width. Considering Coulomb's yield criterion, ground shear interactions T_i can be substituted by $T_i = N_i \tan\phi + c l_i$, where l_i is each sliding surface area per unit width. Water presence is not considered, so ground shear interactions are expressed in total pressures. The rest of the parameters are graphically explained in Figure 13.

Four equilibrium equations are considered, two in each block, where 5 unknowns have to be worked out: Q, N_1, N_2, N_{12}, α_{SD}. In order to obtain the value of *Q*, expression (16), which depends on known parameters k_i and α_{SD}, is maximised, providing the additional equation needed to solve the system. The value obtained Q_{max} is then used to select the specific flexible system (membrane-bolts).

$$Q = \frac{2FG\{(\sin\beta - \cos\beta\tan\phi)[\cos(\beta-\alpha_{SD}) - \sin(\beta-\alpha_{SD})] - \cos\beta[\tan\phi\cos(\beta-\alpha_{SD}) + \sin(\beta-\alpha_{SD})]\} + \cdots}{\tan\phi\sin(\theta+\alpha_{SD}) + \cos(\theta+\alpha_{SD}) + F\sin(\beta+\theta)[\sin(\beta-\alpha_{SD}) - \tan\phi\cos(\beta-\alpha_{SD})] + \cdots}$$
$$\frac{\cdots + FG\sin\alpha_{SD} - G\tan\phi\cos\alpha_{SD} + cl\{F[\tan\phi\sin(\beta-\alpha_{SD}) - \cos(\beta-\alpha_{SD})] - \sec(\beta-\alpha_{SD})\}}{\cdots + F[\cos(\beta+\theta) + \sin(\beta+\theta)\tan\phi][\cos(\beta-\alpha_{SD}) - \sin(\beta-\alpha_{SD})\tan\phi]} \quad (16)$$

$$\frac{dQ(k_i, \alpha_{SD})}{d\alpha_{SD}} = 0 \rightarrow Q = Q_{\max} \quad (17)$$

3.8. Wedge located between two rows of bolts (for rocks)

This model is applied by a company to check the integrity of the membrane under tensile and normal forces regarding a possible rock wedge that may emerge between two rows of anchors. Its field of application is limited to instabilities in rock slopes. It is a model that complements the one presented in section 3.5, providing a complete design methodology of the whole bolt and membrane system (Officine Maccaferri S.p.A., 2006).

The membrane function assumption is to prevent local instabilities in wedges limited by rows of bolts (see Figure 14). Moreover, the hypothesis is based on the idea that the membrane will be unable to exert a pressure normal to the ground, due to the difficulty in applying an appropriate pre-tension and the impossibility of guaranteeing a complete membrane-slope contact. For this reason, the membrane's safety factor is verified under tensile and point loads. The main hypothesis is that the membrane will have to sustain a wedge whose length is defined by the vertical separation between bolts, l_y, with a depth *s,* identical to the one considered in the infinite slope model (see section 3.5). Expression (11) is used again, assuming that *R=0*, and that $\beta = \alpha$, where α is the angle of the joint surface of the local wedge. The force F_{local} acts in the same direction as the joint angle α. W_{local} represents the weight of the local wedge prone to slide.

$$F_{local} = Stab.\ force - Driving\ forces = W_{local}\left[sen\alpha(1 - c - \gamma_{dw}) + c\gamma_{dw}\cos\alpha\right] \qquad (18)$$

4. ANALYSIS OF THE CURRENT METHODOLOGY

Various hypotheses have been established by the different authors and manufacturers to describe their own models. The aim of this section is to verify whether these hypotheses fulfil the reality of flexible systems on site.

4.1. Hypothesis 1: *"Stability analysis applying limit equilibrium"*. (Hypothesis proposed in all models).

In the case of either soils or rock, a static analysis is performed applying force equilibrium at the moment of failure. To provoke the failure, a very tiny shear movement will have to take place in the sliding surface, which produces a certain shear stress that reaches the failure criterion. However, these shear movements are very small, of the order of 1 mm for sands or rock joints (Barton and Choubey, 1985; Bolton 1986) and 1 cm for clays (Skempton, 1985), so that it is necessary that any system intended to prevent sliding, whether it is a flexible membrane or bolts, exerts all the necessary stabilisation force from installation, preventing any movement, even a minimal one.

If these initial little shear movements are not prevented from overcoming the failure limit, which is in practice the most probable case, the system should be designed as a merely passive system. This means that the membrane can only sustain the unstable mass once it has started to slide.

Three situations could theoretically be present on site, depending on the membrane's initial pre-tension force:

– Active membrane with the appropriate pre-tension force and curvature: If the membrane can be installed with a controlled pre-tension force T and the slope presents a parabolic shape with a known mid point deflection f between rows of bolts, then it is possible to apply the design pressure p (see Figure 15). This pressure p would prevent the sliding taking place, so limit equilibrium analysis is a valid design method. In reality, neither membrane pre-tension nor deflection f are measured, hence there is no guarantee of applying the design pressure p to the ground.
– Passive but rigid membrane: if the membrane presents a high initial pre-tension and a convex shape in contact with the whole slope surface (but neither of them are controlled), the unstable mass would start sliding slightly, reaching the failure criterion and continuing to move. If the membrane is highly rigid, the mass would be detained after a few centimetres, developing a very low velocity, and the membrane would undergo low deformation. In this case, limit equilibrium analysis could be used, but considering residual strength (instead of peak strength), leading to a safer solution. On many occasions, membranes are not perfectly rigid and do not present a convex shape. Therefore, limit equilibrium analysis using residual friction angle is not recommended for design, since it does not consider dynamic friction coefficients and large membrane deformations.
– Passive and limitedly rigid membrane: when the membrane has little initial pre-tension, or when it does not present a convex shape, the unstable mass of soil or rock could start sliding at a velocity that can cause the membrane to deform significantly. Therefore, the most adequate analysis would be a dynamic numerical simulation of the interaction membrane - unstable soil/rock - stable slope. This is the most typical and unfavourable case, so dynamic numerical simulation is the design method approach that should be considered.

4.2. Hypothesis 2: *"Membranes are able to exert an evenly distributed normal force over the slope surface increasing the stabilising forces"*. (Hypothesis proposed in models 3.1, 3.2, 3.3, 3.4, 3.6, 3.7. See Figure 15)

Taking into account the data analysed from manufacturers and installers, as well as the different field visits where the installation process has been observed, the conclusion reached is that the pressure that the membrane exerts on installation is not uniformly distributed.

Assuming the membrane has an initial pre-tension, the ground must have a convex curvature of 2nd order (parabola, circumference, catenary, etc) which will apply different pressure distributions. In reality, there are generally two types of slope in terms of geometry: those with a planar surface and those with a more irregular geometry. In the first case, the manufacturers consider that the convexity of the ground is reached thanks to a depression introduced around the bolts, so that the spike plates are below the surface plane (see Figure 16). In the second case, the ground shows isolated protruding points so that the membrane will be able to exert a force on the ground only at these points. In the first case, the force that the membrane could exert would only affect a radius of less than 0.5 m around the bolt (approx.), bearing in mind that the size of the depression made around the bolt head does not usually exceed 15 cm. In the best case, the membrane would cover the ground with a curved shape (see Figure 17), although the membrane could still come into contact with the ground at localised points. In the second case, the membrane would exert a pressure on the ground at isolated points, which are difficult to predict.

In none of the different design methods is the pre-tension force of membrane *T* calculated to obtain the necessary ground stabilization pressure. This pre-tension force *T* would depend on the shape of the curve (parabola, circumference or catenary), its mid point deflection *f*, and the separation between bolts *l* (see Figure 17).

During the installation process, the pre-tension force to which the membrane is submitted is not controlled. In addition, the precise depression of the bolt heads is not measured. Moreover, it would be impossible to know at which points the membrane exerts pressure on the ground when the slopes have an irregular surface.

4.3. Hypothesis 3. *"An appropriate tightening of bolts can prevent sliding of the unstable layer, either soil or rock, by increasing the stabilising forces"*. (Hypothesis proposed in models 3.1, 3.2, 3.3, 3.4, 3.6, 3.7)

Bearing in mind the data from manufacturers and installers, as well as from the different field visits in which cable nets have been installed, the conclusion reached is that bolts are not tightened with the designed force, since it is rarely measured. The three manufacturing companies considered in this article use a torque wrench with an arm of about 50 cm, in which the force exerted is generally not controlled. In few cases do installers use a dynamometric wrench to verify the torque applied, estimating that the compression force exerted on the bolt is around 50 kN. To obtain a better idea of the magnitude of the necessary force to exert on a bolt in order to stabilise an unstable layer of 1 m thickness, on a 40 m high slope, with an inclination angle of 60º, ϕ=30º, c=10 kN/m2, γ=16 kN/m3, without water and with bolts perpendicularly bolted 4 m apart, we would need a force of 160 kN per bolt, without applying any partial safety coefficient to the parameters.

A point load applied on the ground surface will be transformed into a non-uniform, depth-dependent pressure distribution according to the Boussinesq theory (1885). Figure 18 shows the distribution of vertical pressures at 1 m depth due to a point force of 50 kN, regarding radial symmetry. The x axis represents the distance to the axis of force application. This non-uniform pressure implies that for large spacing between bolts, the ground at a distance from these is receiving a very reduced pressure.

4.4. Hypothesis 4. *"The membrane transmits an upward shear force to the ground as a consequence of the initial pre-tension to which the membrane is submitted".* (Hypothesis proposed in model described in paragraph 3.6)

When exerting a pre-tension on the membrane, a compression force would be achieved parallel to the slope, but not a shear upslope force. Moreover, the compression force in the same direction as the slope would not imply an increment in the stabilising forces preventing sliding, because it would not work perpendicularly to it.

4.5. Hypothesis 5. *"Failure mechanism consisting in wedges separated between rows of bolts"* (Hypothesis proposed in the models described in sections 3.3, 3.6, 3.7).

Different authors and companies propose failure mechanisms based on soil or rock wedges limited by rows of bolts. This failure mechanism is based on the idea that bolts can in some way induce the breakage of the ground. However, this failure mechanism has neither been theoretically demonstrated nor has it been observed on site.

In the case of soil slopes, what has been observed is that once sliding is produced, the horizontal reinforcement cables, due to their initial pre-tension, may be able to hold the material. Thus, pockets of ground can be seen that are limited in their lower part by a horizontal reinforcement cable. However, the calculation approach to model this situation should be based on numerical simulations using a model that studies the interaction between membrane - unstable soil/rock - slope.

4.6. Hypothesis 6. *"In a failure mechanism defined by wedges, the part of the ground above an unstable wedge is stabilised by the membrane and bolts".* (Hypothesis proposed in the models described in sections 3.6, 3.7, 3.8).

Firstly, none of the authors accurately define the hypothesis, because it is not sufficient to state that the higher ground is stabilised by the membrane and bolts. It is necessary to indicate that there must be a crack in the upper wedge edge where there is no kind of interaction with the higher ground. Thus, the equilibrium equations that are established can be solved, because otherwise, there would be more unknowns than equations.

On the other hand, the stabilization pressures calculated by this mechanism are lower than those assuming infinite slope. This implies that unless it is reliably known a priori that this will be the failure mechanism, the hypothesis of infinite slope would be preferable to be on the safe side.

5. ON SITE PERFORMANCE vs. DESIGN: PRESSURE COMPARISON

The aim of this section is to compare the theoretical normal force transmitted to the ground in order to prevent any sliding vs. the real one applied in terms of installation procedure. In this section, the membrane is considered to be able to exert a certain pressure normal to the ground if the ground has a convex shape and the membrane has a certain pre-tensioned force. In relation to bolts, the tightening force applied does not exceed 50 kN, according to a certain manufacturer (Geobrugg Ibérica 2008).

There are two different ways to transmit this force to the ground. If the depression around the bolts is very small, the torque applied on the bolts will be transmitted to the spike plate and then mainly to the ground (see Figure 19, Case 1). The second way considers the situation when there is a deep depression around the bolts (see Figure 19, Case 2), hence the torque on the nut will be transmitted to the spike plate and then mainly to the reinforcement cable by exerting a tensile force T. This force T, considering that the cable has a convex shape, will be transmitted to the ground as a distributed pressure p. The component of that pressure p normal to the ground should coincide with the total axial force applied to the bolt (50 kN). Thus, independently of how the force is transmitted to the ground, the total normal force applied to the slope surface is around 50 kN/bolt, and it does not depend on membrane typology.

Company 1 proposes a cylindrical model for membrane deformation between rows of bolts (see Figure 19 and Figure 20), so the normal force that the membrane can exert on the ground depends on vertical spacing between bolts Sy (Luis Fonseca 2010), but not on Sx. Typical values of horizontal separation between bolts, Sx, are listed in order to calculate p_{REAL} as the tightening force, 50 kN, divided by the bolt spacing ($Sx \cdot Sy$). In Table 1, a comparison between theoretical pressures calculated by a manufacturer for different membrane arrangements (p_{DESIGN}) vs. the real ones (p_{REAL}), taking into account installation procedure, is shown. All values have been calculated with a safety factor of 1.0. In the most favourable case, real pressure on site was less than 14% of the design pressure.

Company 2 provides a design table for each specific solution in their technical brochure (MTC 2004). In this case, pressure design is calculated considering a spherical model of membrane deformation, so both Sy and Sx define the normal pressure that the cable membrane exerts on the ground in this case. In Table 2 a comparison between design and real values is shown. All values have been calculated with a safety factor of 1.0. Real pressure on site was less than 19% of the design pressure in the most favourable case.

Company 3 uses contradictory terms to define the behaviour of these systems. In its technical brochure, they describe these systems as *'active'*, because they can prevent sliding (Officine Maccaferri S.p.A. 2008). However, in their help manual from their design software the company assumes *'passive behaviour'* of both bolts and geomembrane (Officine Maccaferri, 2006). This manufacturer does not provide specific design tables in order to select a specific product solution in relation to the desired stabilisation pressure, so there is no possibility of comparing design table values with real ones. Assuming the correctness of what it is stated in the help manual of their design software, the company postulates that these systems are passive. Although this description approximates better to the real behaviour of the membrane and the bolts, the design methods they used lack a rigorous approach.

In relation to bolt design (see paragraph 3.5), limit equilibrium analysis is applied again, even though passive behaviour of bolts has been assumed. In addition, a non-cohesive Coulomb criterion with a constant friction angle of 45º has been assumed for any case, leading to an unsafe calculation in very polished rock joint surface or in joints filled with soil. Furthermore, it was assumed that maximum allowable yield tensile stress is applied on the bolts, which is not true in every case.

Regarding membrane design (see paragraph 3.8), the third company considers that a force is applied to a rock wedge which is not stabilised by shear resistance at the joint surface. Therefore, even though the behaviour of the membrane is assumed to be passive, limit equilibrium analysis is used to calculate the force that the wedge exerts on the membrane. The membrane is a limitedly rigid system so a dynamic numerical approach should be considered for analysis of the interaction between membrane – wedge –slope with a dynamic friction coefficient between stable slope and unstable wedge, instead of limit equilibrium analysis. It is important to remark that real forces acting on the membrane could be significantly higher than those calculated by the third company which assumed that the friction angle between wedge and stable slope could have lower values than *tan 45º* as stated in sections 3.5 and 3.8.

In their PhD theses, researchers such as Daniel Castro and Almudena da Costa only define the procedure to calculate the pressure normal to the ground needed to prevent sliding. In tables and graphs, they indicate different values of pressure in relation to geometrical and ground strength parameters, but they do not provide design tables linked to different market products, so comparison is not possible.

6. CONCLUSIONS

Eight different design methods proposed by manufacturers and independent researchers have been described and analysed (Table 3). There is no evidence of the existence of any numerical methodology to design these systems, since all models found in public bibliography are analytical. Most of manufacturers and independent researchers assume active behaviour of these systems when they propose their design models. In all the existing design models limit equilibrium analysis is considered with a particular failure mechanism, either wedge shape or infinite slope. A uniform pressure p normal to the ground is calculated in order to increase the normal effective stress on slope surface and therefore the shear resistance in the potential sliding surface. However, the hypothesis of active behaviour has not been demonstrated by any company designer or independent researcher.

There are two main conditions that any membrane system should fulfil in order to prevent rock or soil sliding in an *active way*. If either of these two conditions is not present, the system is not active and would behave as a passive one, which means that it would retain a mass of ground once the instability has already occurred.

- The ground section must have convex curvature, so that the membrane may transmit a uniformly distributed pressure to the ground, which will project the internal tensile stresses that the membrane will induce due to its pre-tension. This condition is essential for the membrane to exert an evenly distributed pressure normal to the ground.

- The membrane must be pre-tensioned before the fastening of its ends and intermediate points, with a force that will depend on the pressure p necessary to stabilise the ground, the type of convex 2nd order curve (circumference, parabola, catenary, etc.) and the mid point deflection of the curve f (see Figure 17).

In relation to the first condition, in most cases, slopes are planar with small depressions around the bolt zone, with the aim of giving some convexity to the ground. However, in reality, the membrane shape is a kind of trapezoid with rounded vertices (see Figure 16), where the membrane's pre-tension force might be transmitted to the ground in the zone around the bolts, this force being null elsewhere.

Regarding the second condition, analyzing the installation systems of the different manufacturers, it has been observed that they do not measure the pre-tension force applied to the membrane. Tightening force on bolts is measured, but only on certain occasions, achieving a value of 50 kN. Bolt tightening is the only force that can be considered to contribute to the overall slope stability. Design and real forces were compared in section 5, finding that, in the most favourable case, real pressure is less than 19% of design pressure.

The main conclusion of this review is that flexible systems anchored to the ground are not active; therefore, they can only contain an unstable mass once it has already started to slide. Current design methods are based on a limit equilibrium analysis, which is more appropriate when active behaviour does exist. Therefore, design methods employed nowadays are not adequate, leading to an extra installation cost in the flexible system in some cases or to an unsafe solution in others.

A new design methodology is also recommended by the authors. This new approach considers that the membrane should be designed in order to contain a mass of material that has already started to slide.

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Figures

Figure 1.- Cable net.

Figure 2.- High-resistance wire mesh.

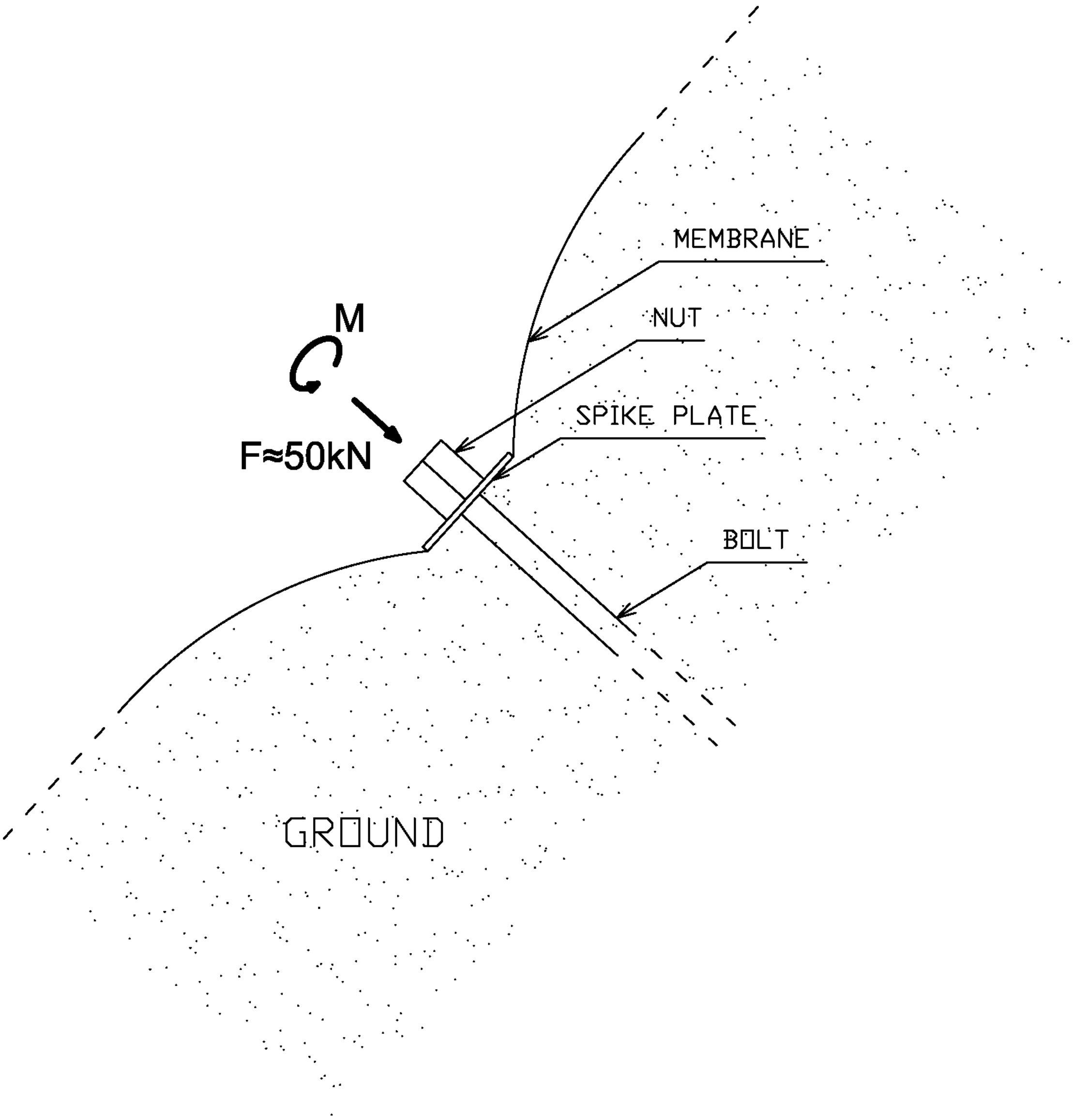


Figure 3.- Approximate tightening force on bolts (Geobrugg, 2008).

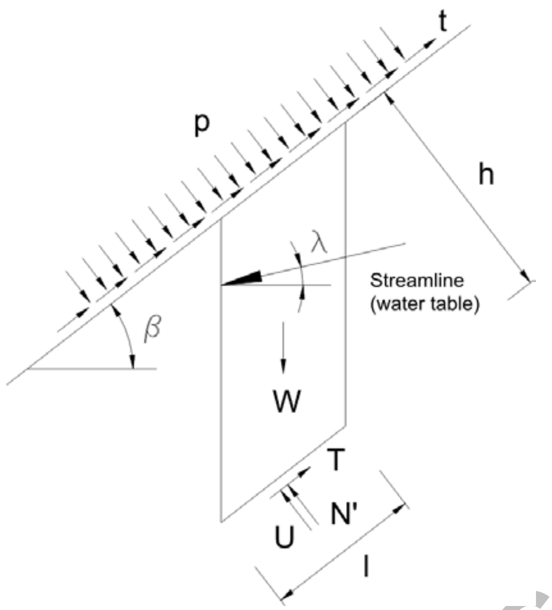


Figure 4.- Infinite slope (for soils, da Costa A.).

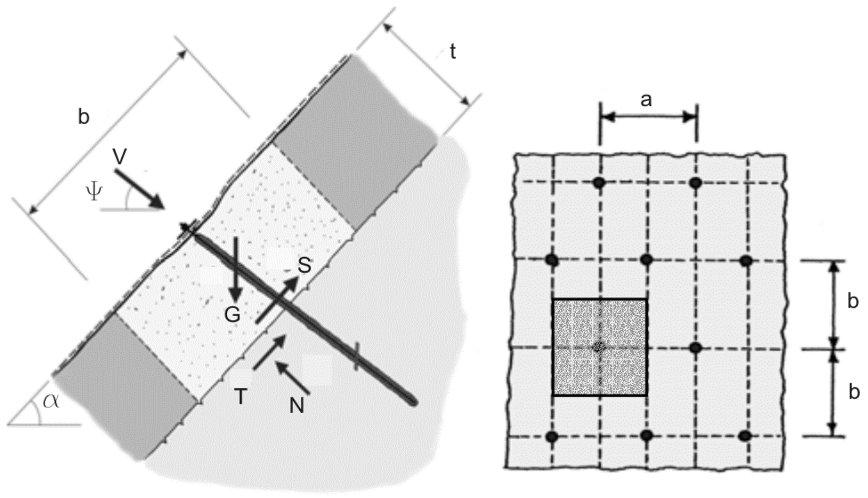


Figure 5.- Infinite slope (for soils). Model proposed by manufacturer (Guasti, 2003; Flum, 2004).

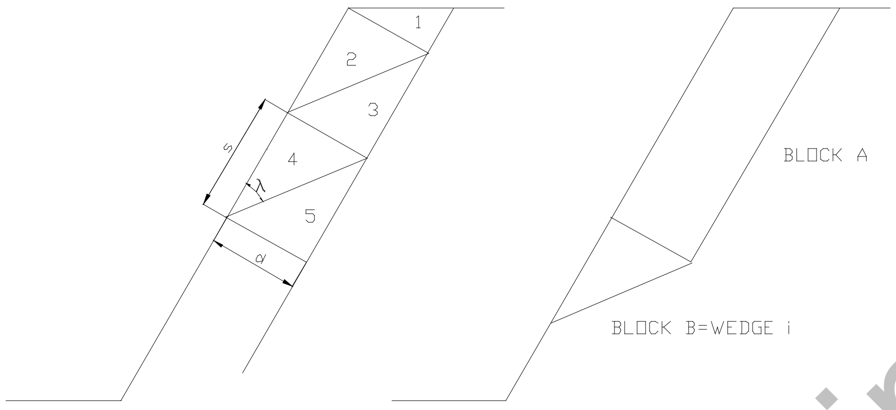


Figure 6.- Slope discretised in wedges (for soils).

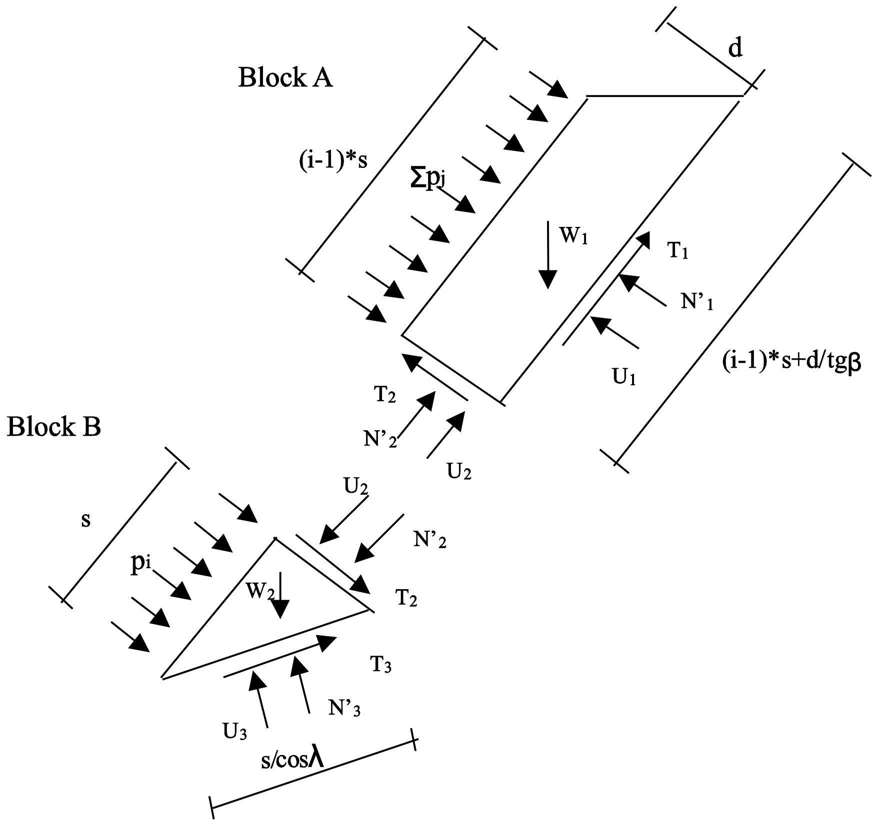


Figure 7.- Slope discretised in wedges (for soils). Force scheme.

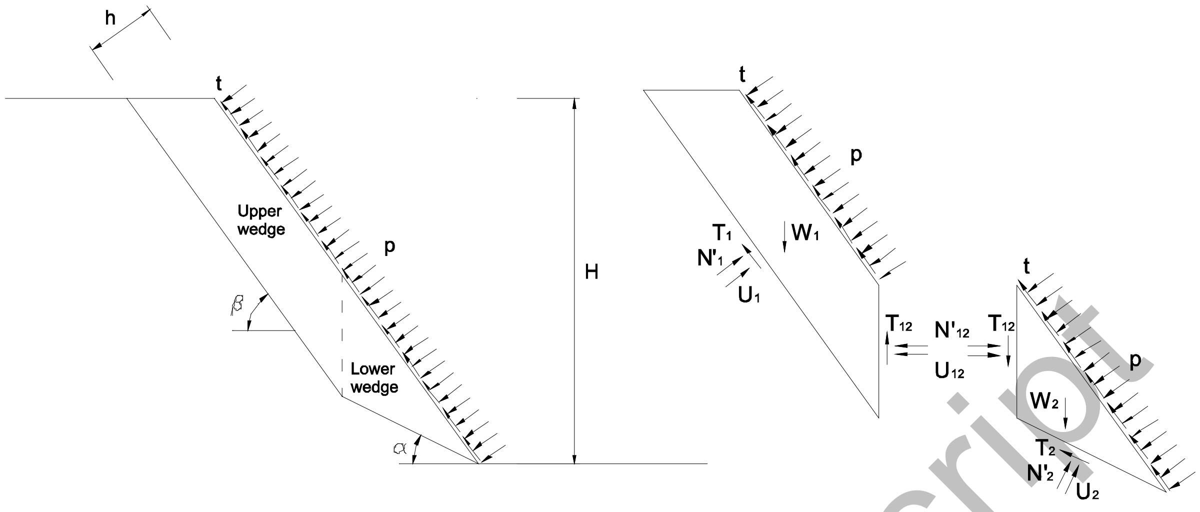


Figure 8.- Slope discretised in block and wedge (for soils).

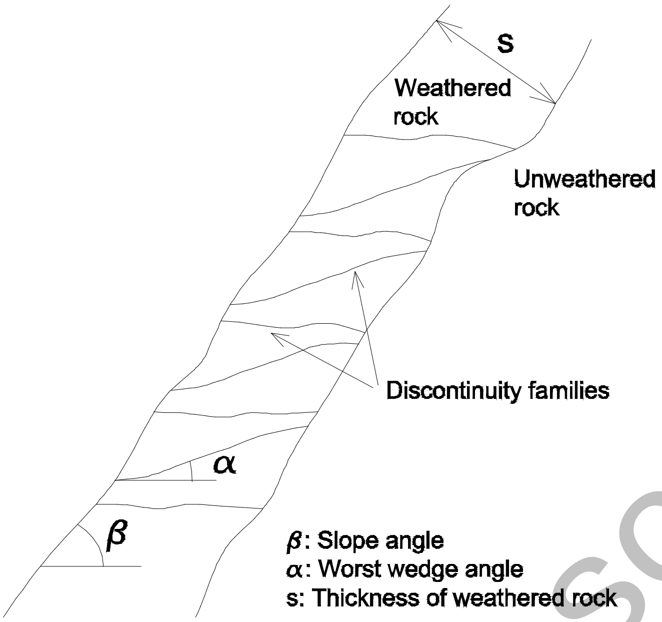
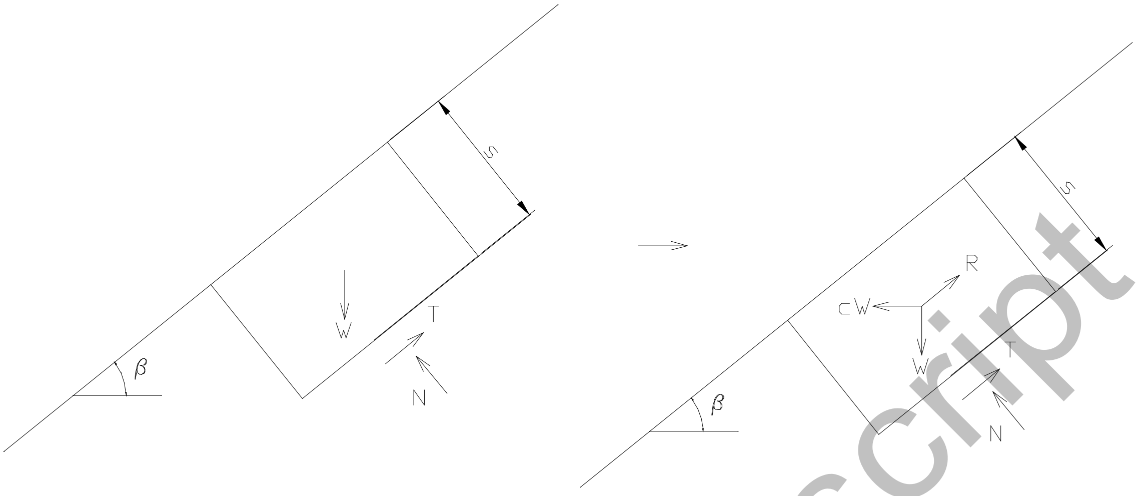


Figure 9.- Instabilities in rock slope.



FoS=1
Stabilising forces=Driving Forces=Wsenb

$$FoS=\frac{\text{Stab. forces}}{\text{Driving forces}}\approx\frac{W sen\beta - cW sen\beta \tan\Phi + R}{\gamma_{dw} W(sen\beta + c\cos\beta)}$$

Figure 10.- Infinite slope (for rock).

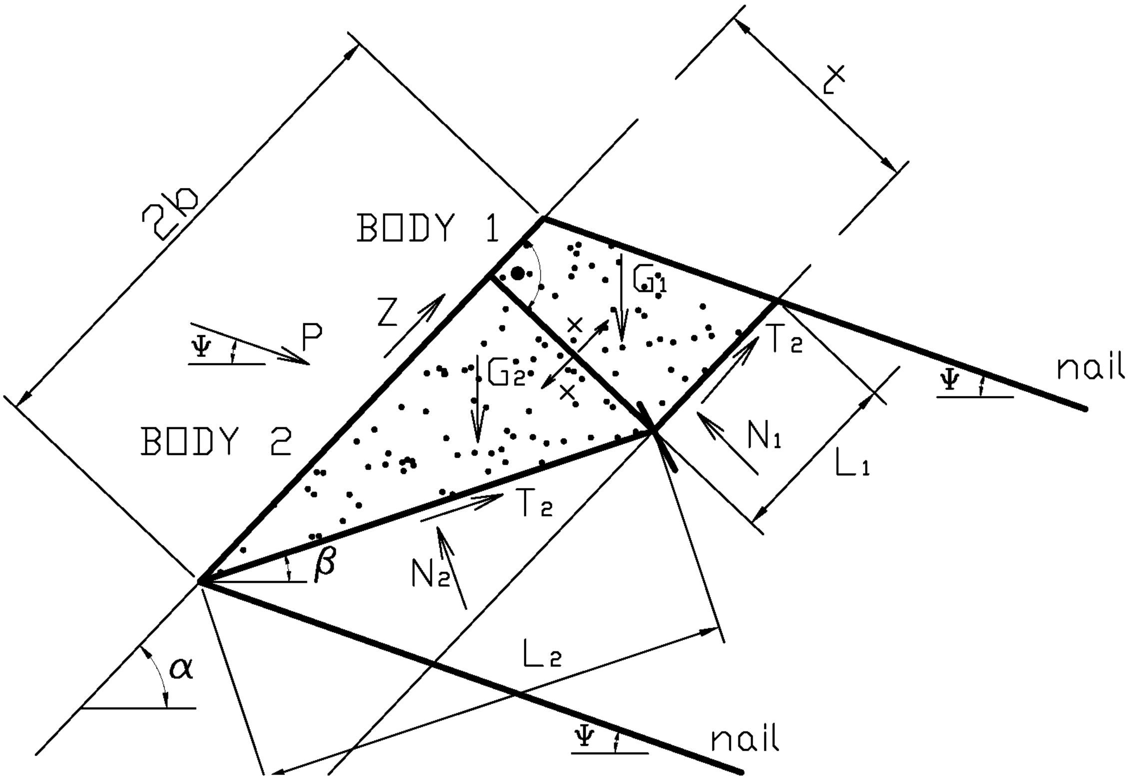


Figure 11.- Block and wedge limited between two rows of bolts (for soils, Geobrugg).

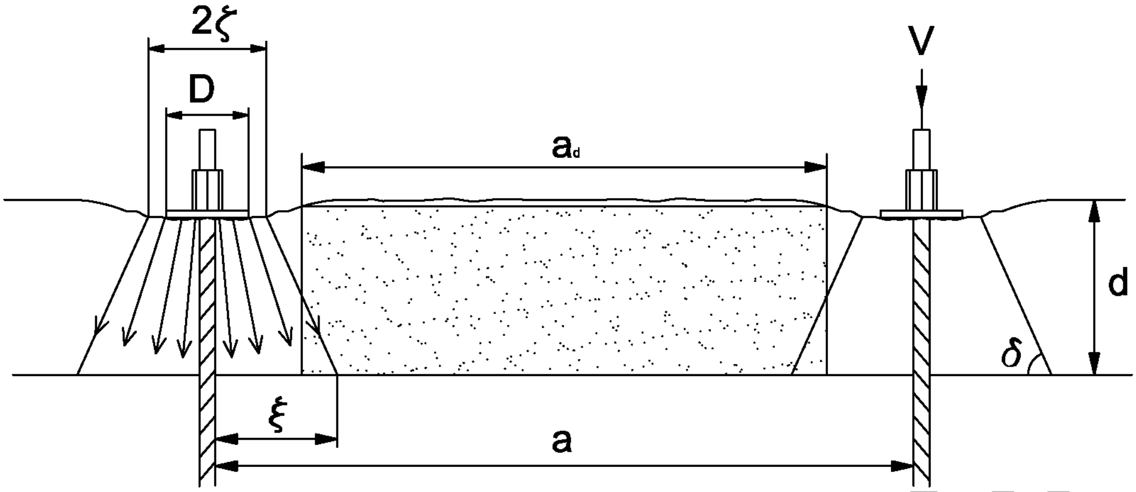


Figure 12.- Width of an unstable soil wedge (Yang, 2006).

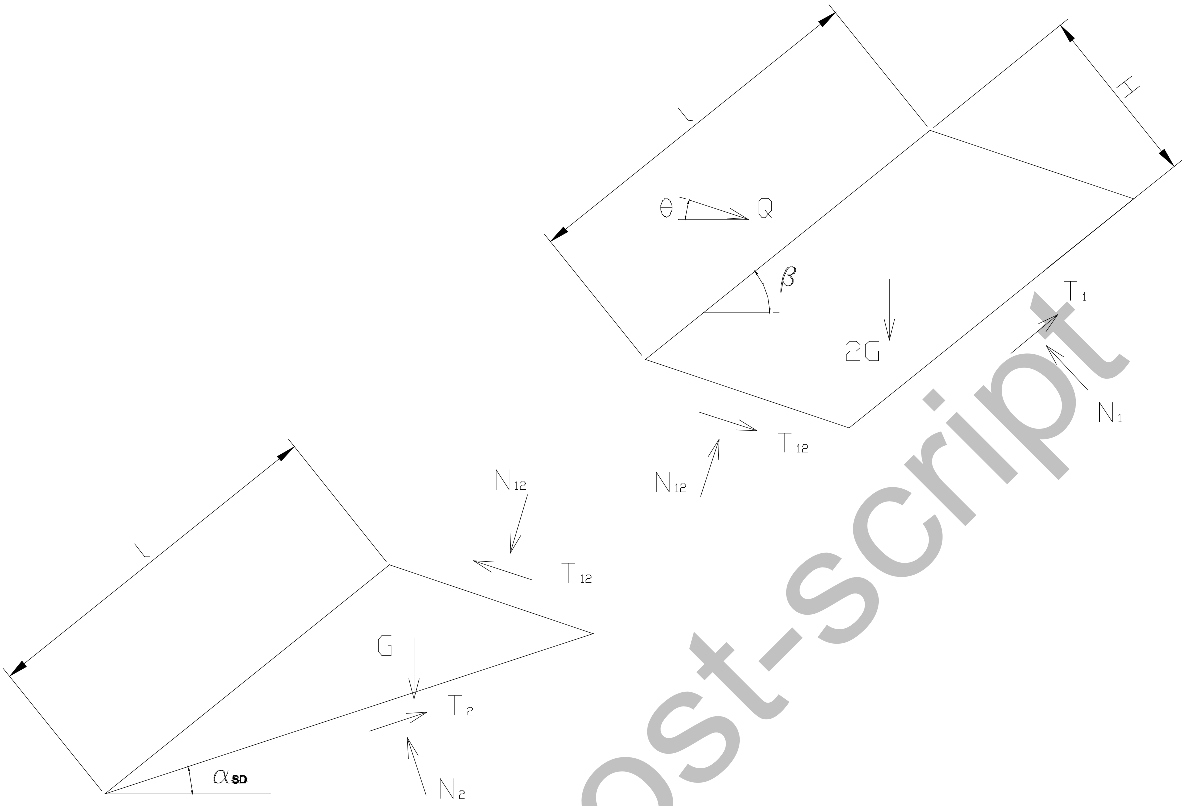
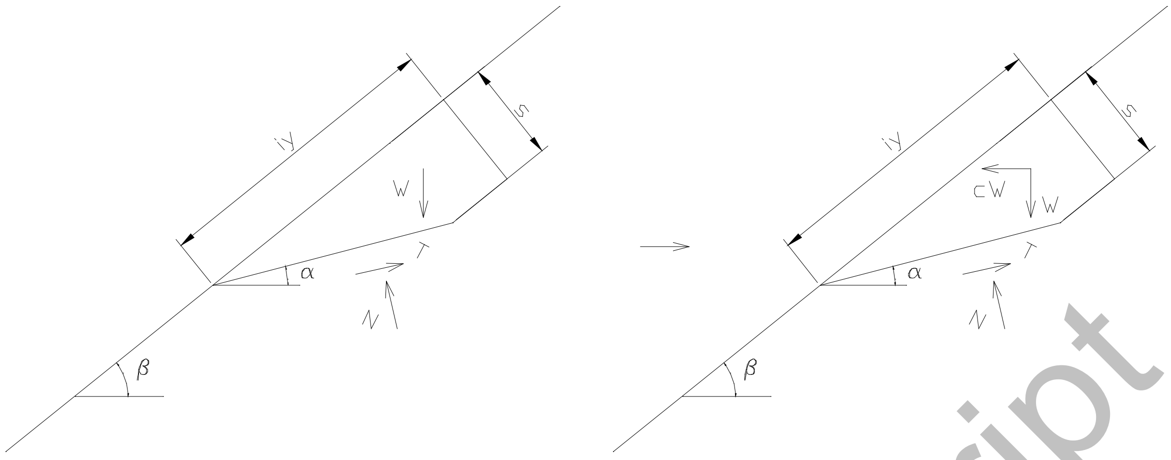


Figure 13.- Block and wedge limited between two rows of bolts (for soils, Castro D.).



FoS=1
Stabilising forces=Driving Forces=W$\text{sen}\alpha$

F_{local} = Stab. force - Driving forces = $W_{local}\,[\text{sen}\alpha(1\text{-}c\text{-}\gamma_{dw})+c\gamma_{dw}\cos\alpha]$

Figure 14.- Wedge located between two rows of bolts (for rock).

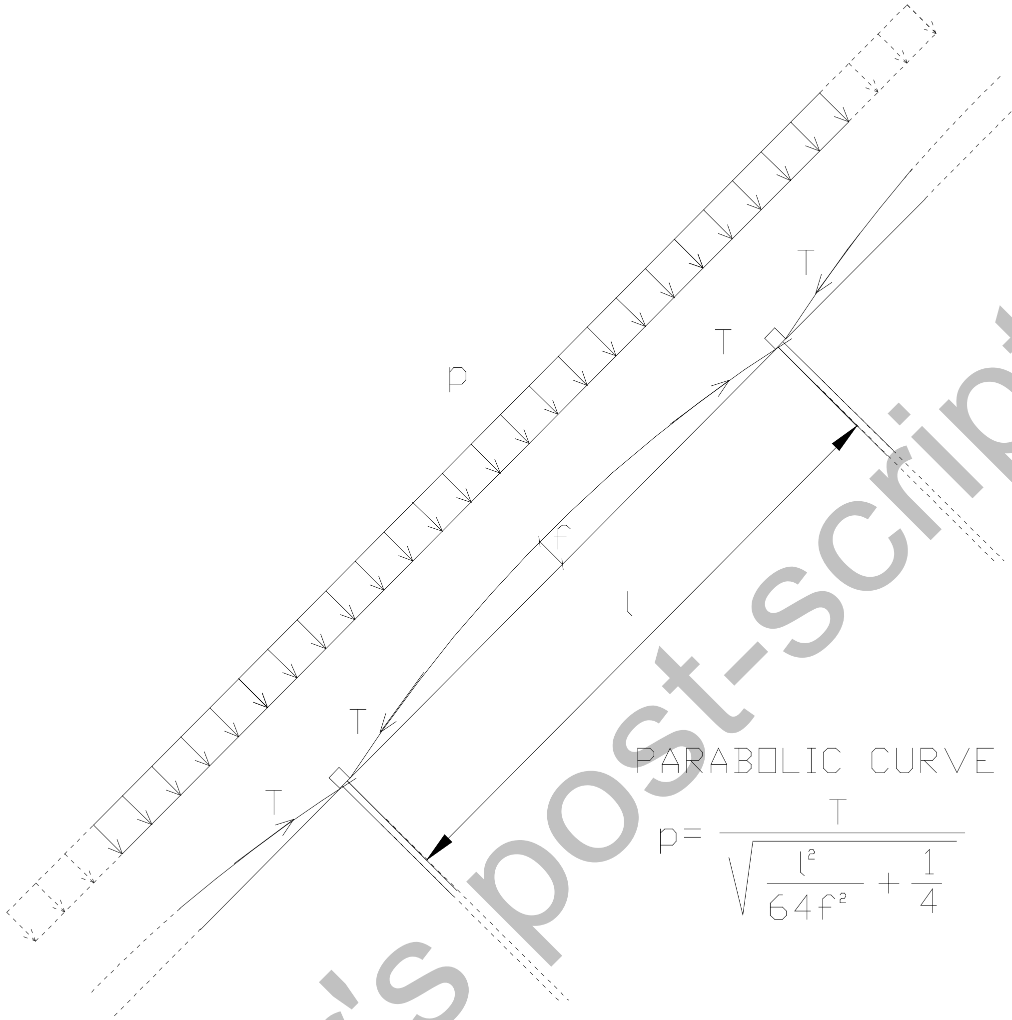


Figure 15.- Pressure exerted membrane-ground. Theoretical situation (simplification in 2D).

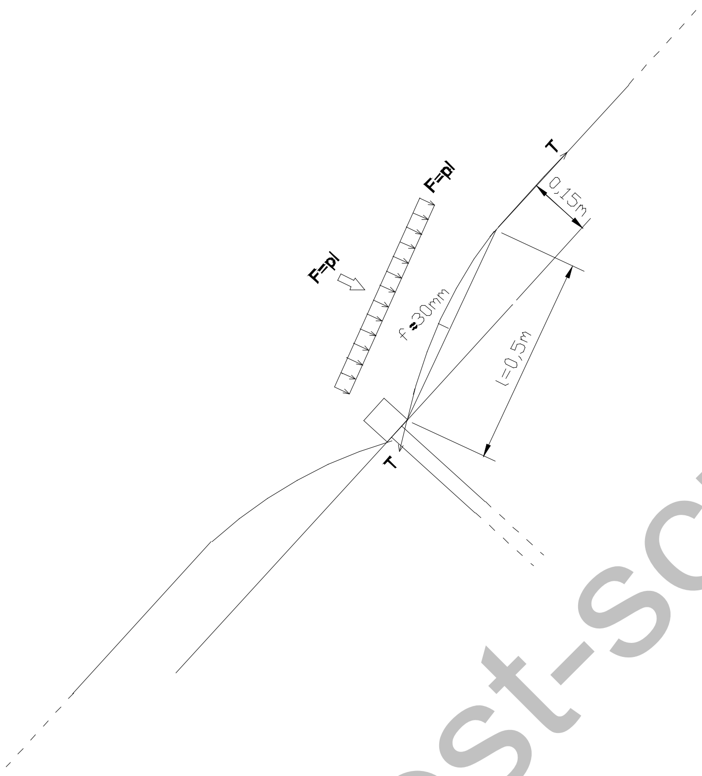


Figure 16.- Pressure exerted membrane-ground. Actual situation (simplification in 2D).

PARABOLIC CURVE

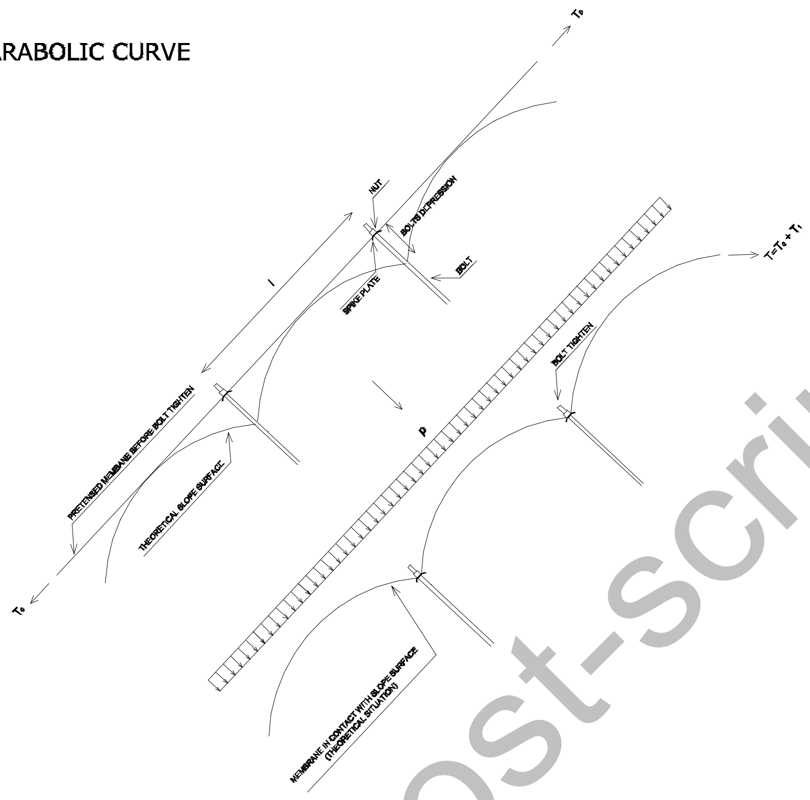


Figure 17.- Effective process of pre-tension and fixing of the membrane (2D scheme).

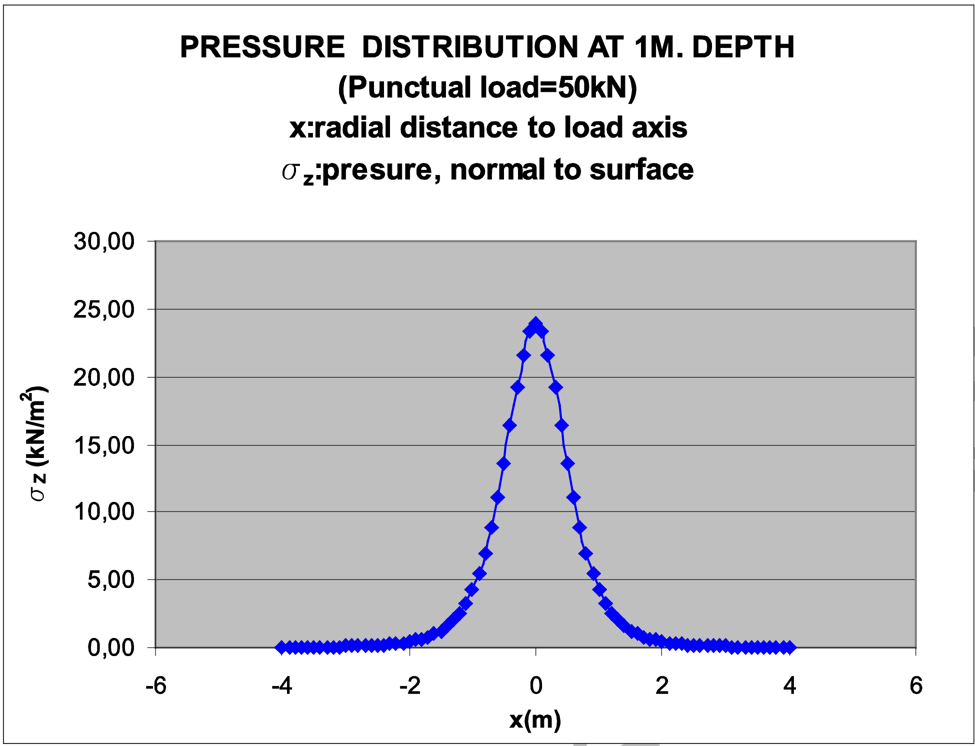


Figure 18.- Vertical pressure distribution at 1 m depth (point load). Boussinesq theory.

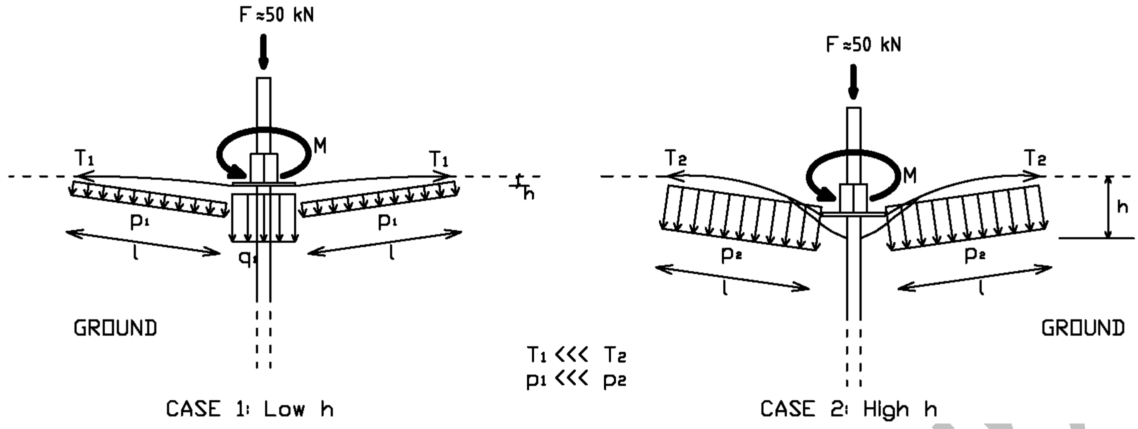


Figure 19.- Bolt-ground force transmission mechanisms.

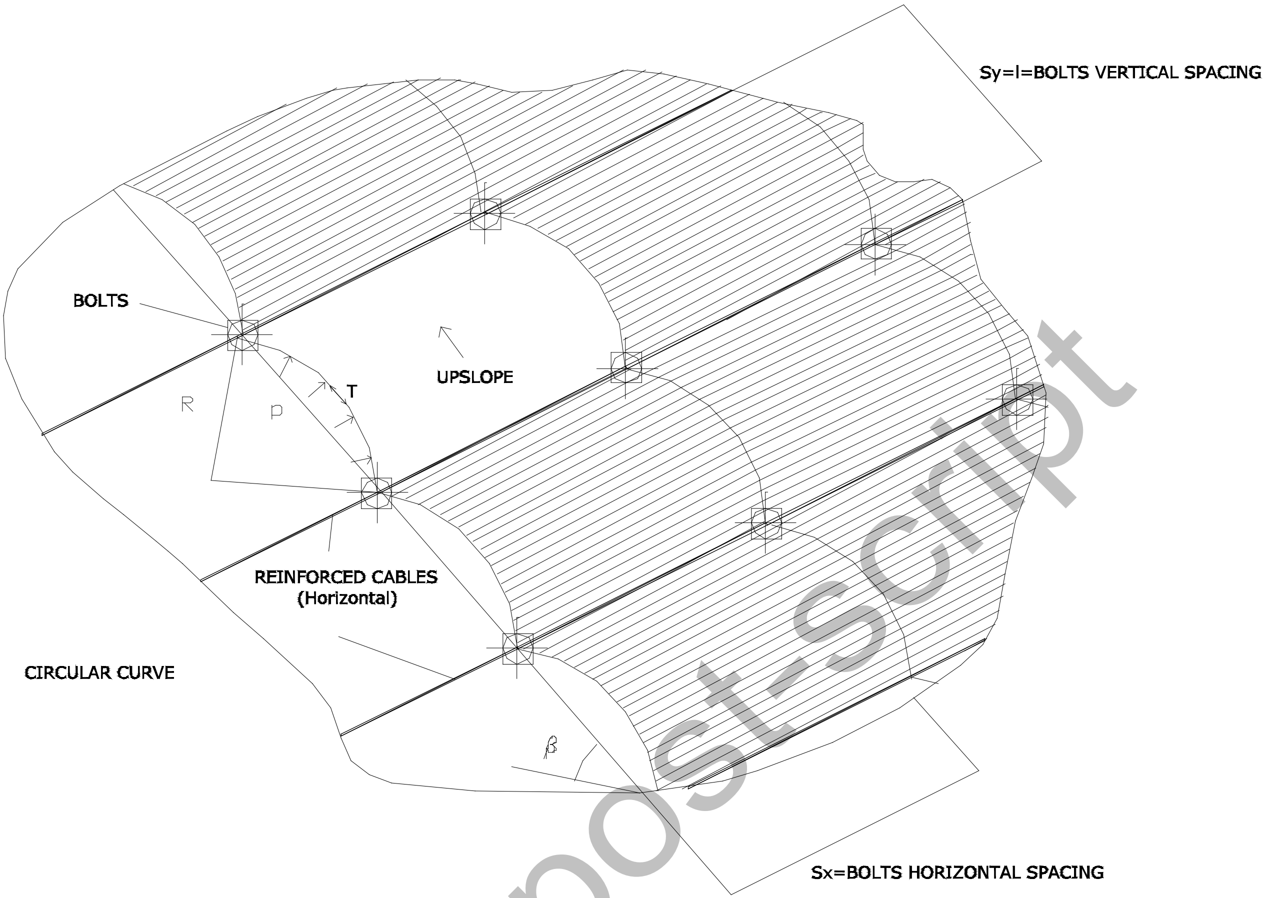


Figure 20.- Membrane deformation. Cylindrical model.

Tables

Table 1: Wire mesh. Design pressure vs. real pressure. Company 1.

PRODUCT	Sy (m)	Sx (m)	p_{DESIGN}(kN/m2)	p_{REAL}(kN/m2)	p_{REAL}/p_{DESIGN}(%)
S-5	3	3	40.1	5.6	13.9
S-10	3	5	40.1	3.3	8.3
S-15	2.5	4	55.1	5.0	9.1
S-20	2.5	5	55.1	4.0	7.3
S-30	2.5	5	55.1	4.0	7.3
S-40	2.5	4	55.1	5.0	9.1

Table 2: Cable net. Design pressure vs. real pressure. Company 2.

GRID SIZE	Sy (m)	Sx (m)	p_{DESIGN}(kN/m2)	p_{REAL}(kN/m2)	p_{REAL}/p_{DESIGN}(%)
300 mm x 300 mm	2	2	66.1	12.5	18.9
	3	3	45.0	5.6	12.3
	4	4	33.8	3.1	9.2
	3	5	42.9	3.3	7.8
250 mm x 250 mm	2	2	80.8	12.5	15.5
	2	3	79.5	8.3	10.5
	2	4	76.1	6.3	8.2
	3	3	68.5	5.6	8.1
	3	4	53.2	4.2	7.8
	3	5	50.0	3.3	6.7
	3	6	49.0	2.8	5.7
	4	4	43.8	3.1	7.1
	4	8	39.1	1.6	4.0
200 mm x 200 mm	2	2	107.6	12.5	11.6
	3	3	79.6	5.6	7.0
	4	4	53.9	3.1	5.8
	3	5	61.3	3.3	5.4
150 mm x 150 mm	2	2	161.8	12.5	7.7
	3	3	106.9	5.6	5.2
	4	4	83.0	3.1	3.8
	3	5	97.7	3.3	3.4

Table 3: Existing design models.

Model	Author	Element design	Behaviour assumption (1)	Design method	Failure mechanism	Application	Water	Seismicity
1	Da Costa, A.	Membrane	Active	Analytical. Limit Equilibrium Analysis	Infinite	Soil. Shallow instabilities. Overall failure (2).	Yes	No
2	Geobrugg	Membrane	Active	Analytical. Limit Equilibrium Analysis	Infinite	Soil. Shallow instabilities. Overall failure.	No	No
3	Da Costa, A.	Membrane	Active	Analytical. Limit Equilibrium Analysis	Discretised in wedges (wedges limited between bolt rows)	Soil. Shallow instabilities. Overall failure.	Yes	No
4	Iberotalud	Membrane	Active	Analytical. Limit Equilibrium Analysis	Block+wedge	Soil. Shallow instabilities. Overall failure.	No	No
5	Maccaferri	Bolts	Passive	Analytical. Limit Equilibrium Analysis	Infinite	Rock. Shallow instabilities. Overall failure.	No	Yes
6	Geobrugg	Membrane	Active	Analytical. Limit Equilibrium Analysis	Block+wedge (between 2 bolt rows)	Soil. Shallow instabilities. Local failure (3).	No	No
7	Castro, D	Membrane	Active	Analytical. Limit Equilibrium Analysis	Block+wedge (between 3 bolt rows)	Soil. Shallow instabilities. Local failure.	Yes	No
8	Maccaferri	Membrane	Passive	Analytical. Limit Equilibrium Analysis	Block+wedge (between 2 bolt rows)	Rock. Shallow instabilities. Local failure.	No	Yes

(1) According to author
(2) Overall failure: affects to all slope height, even though shallow instability is considered
(3) Local failure: affects only to a certain part of all slope height